This is a digital copy of a book that was preserved for generations on library shelves before it was carefully scanned by Google as part of a project to make the world's books discoverable online.

It has survived long enough for the copyright to expire and the book to enter the public domain. A public domain book is one that was never subject to copyright or whose legal copyright term has expired. Whether a book is in the public domain may vary country to country. Public domain books are our gateways to the past, representing a wealth of history, culture and knowledge that's often difficult to discover.

Marks, notations and other marginalia present in the original volume will appear in this file - a reminder of this book's long journey from the publisher to a library and finally to you.

Usage guidelines

Google is proud to partner with libraries to digitize public domain materials and make them widely accessible. Public domain books belong to the public and we are merely their custodians. Nevertheless, this work is expensive, so in order to keep providing this resource, we have taken steps to prevent abuse by commercial parties, including placing technical restrictions on automated querying.

We also ask that you:

+ *Make non-commercial use of the files* We designed Google Book Search for use by individuals, and we request that you use these files for personal, non-commercial purposes.
+ *Refrain from automated querying* Do not send automated queries of any sort to Google's system: If you are conducting research on machine translation, optical character recognition or other areas where access to a large amount of text is helpful, please contact us. We encourage the use of public domain materials for these purposes and may be able to help.
+ *Maintain attribution* The Google "watermark" you see on each file is essential for informing people about this project and helping them find additional materials through Google Book Search. Please do not remove it.
+ *Keep it legal* Whatever your use, remember that you are responsible for ensuring that what you are doing is legal. Do not assume that just because we believe a book is in the public domain for users in the United States, that the work is also in the public domain for users in other countries. Whether a book is still in copyright varies from country to country, and we can't offer guidance on whether any specific use of any specific book is allowed. Please do not assume that a book's appearance in Google Book Search means it can be used in any manner anywhere in the world. Copyright infringement liability can be quite severe.

About Google Book Search

Google's mission is to organize the world's information and to make it universally accessible and useful. Google Book Search helps readers discover the world's books while helping authors and publishers reach new audiences. You can search through the full text of this book on the web at `http://books.google.com/`

DUPL
C 421,555

Engin. Library

MEMOIRS

OF THE

TÔKIÔ DAIGAKU

(UNIVERSITY OF TÔKIÔ)

No. 11.

A SYSTEM OF IRON RAILROAD BRIDGES FOR JAPAN

BY

ohn lexander ow

J. A. L. WADDELL, C.E., B.A.Sc., Ma.E.,

PROFESSOR OF CIVIL ENGINEERING IN THE UNIVERSITY OF TOKIO, JAPAN: CONSULTING ENGINEER FOR THE FIRM OF RAYMOND AND CAMPBELL, BRIDGE-BUILDERS OF COUNCIL BLUFFS, IOWA: MEMBER OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, RENSSELAER SOCIETY OF ENGINEERS, ENGINEERS' CLUB OF PHILADELPHIA, AND WESTERN SOCIETY OF ENGINEERS; ASSOCIATE MEMBER OF THE INSTITUTION OF CIVIL ENGINEERS, LONDON: AUTHOR OF THE DESIGNING OF ORDINARY IRON HIGHWAY BRIDGES.

(TEXT.)

PUBLISHED BY TÔKIÔ DAIGAKU

TÔKIÔ:

2545 (JAPANESE ERA)

1885 A. D.

Printed by "SEISHI-BUNSHA," Tokio Japan.

TABLE OF CONTENTS.

134962

LIST OF TABLES.

LIST OF PLATES.

CHAPTER I.

INTRODUCTION.

To the Civil and Mechanical Engineers of Japan.

Gentlemen,

Before entering upon the subject of this treatise, it will be necessary to make so many explanations and statements of facts that it will be much better to address you directly; I hope therefore, that you will pardon me for writing this chapter in the first person.

Probably the first question which enters the mind of one who is about to read a technical work is "for what purpose and in whose interest is this book written." The Japanese are generally, and perhaps with reason, suspicious of any proposed innovation by a foreigner, thinking that the proposer may have an "axe to grind." On this account they occasionally fail to profit by the experience and advice of those who really have the interests of Japan at heart. But as nothing is done without a reason, it may be as well to explain, before going any further, why this book was written.

In the first place let me assure you, gentlemen, that *I have not an axe to grind*; because my stay in this country is for various reasons necessarily limited to a little over a year longer; indeed, it has been with difficulty that I have so arranged my family affairs as to be able to comply with the request of the University authorities to remain a year longer than my original contract stipulated; consequently I can in no way be pecuniarily benefitted by your adopting the system of bridges herein proposed. My reason for preparing the treatise is simply this.

I came some two years and a half ago to Japan, hoping not only to be at the head of a large department, but also to be able to occupy my spare time in attending to practical engineering work. Instead, I have found that there is no work in the country for foreign engineers; and, what is worse, that there are never more than a dozen students in the engineering department. Now as I am unwilling to depart from Japan after a sojourn of three or four years without leaving behind me some professional record of my stay, I have devoted a twelvemonth of my spare time to the preparation of this work, which I hope will meet with your approval.

In suggesting that you change your present style of bridge designing to that expounded in the following pages, it is no untried experiment that I am asking you to make; for the system proposed is essentially in agreement with the best American practice in bridge construction.

That the United States of America lead the world in bridge building is a fact undisputed even in Europe. It is no wonder that such is the case, for owing to the immense extent of territory and the rapid progress in railroad building of late years, there have been more iron bridges built there than in any other country. Then again the amount of capital available for railroad purposes or any other engineering work being much less in the United States than that which could be obtained in older and richer countries, and the cost of all kinds of labour being very high, it has been found necessary in all branches of construction to study economy. In no department of engineering is this fact more evident than in bridge building, for not only is it made a specialty by many companies, but what is more, when American bridges are put in competion with those of other countries, the American bridges are chosen, notwithstanding the higher prices of American labour and materials.

In this connection let me quote a little from an article which appeared some eight months ago in the *New York Times* upon "Bridge Building in America." "If there is anything in which the United States can justly claim precedence over all other countries it is for the simplicity, mechanical construction and boldness of design of their bridges." This remark was made to a Times' reporter, and with a good deal of pride, by Mr. Thomas C. Clarke, of the Union Bridge Company, and one of the veteran bridge-builders of the company. "The Brooklyn bridge," he added, "ha s the largest span and is considered the largest bridge in the world. But we will soon be obliged to yield the palm of having the biggest bridge to another country. There is now building over the Firth of Forth, in Scotland, a bridge of two spans, each of which is as long as the Brooklyn bridge. This is the greatest bridge ever designed anywhere. We are up to nothing of the kind in America, and we haven't money enough for it."

"There are probably 300 miles of iron bridges in the country now, and perhaps in the neighborhood of 700 miles of wooden bridges. I am speaking now of railway bridges. The construction of road bridges is quite a separate and distinct industry. It is the price of iron that regulates the cost of a bridge; the cost of labor has very little to do with it."

"To come back to bridges," continued Mr. Clarke, all, or nearly all, the steel used in railway bridges is made here, very little being imported. That new bridge at Rondout, on the west Shore, if built ten years ago, would have been the subject of a book. Now it is simply a railroad bridge, and not one traveller in ten even looks at it as he goes over it. It is very light, yet perfectly secure. That is a great point where American engineers excel—in having lightness combined with perfect security. It is a saving to the railways, too, for bridges are paid for by the pound. Now an order is given for a bridge just as it is for a locomotive—it is mere matter of commercial manufacture. When I was in England some years ago I wanted to go and see the Tay bridge, but the civil engineers said: 'O, that's not much good; it's not worth going to see.' I didn't see the bridge. But I know its construction was so palpably erroneous that a common house carpenter could have seen its unsafe condition. Our American railway history shows nothing the equal of that great disaster, though the Ashtabula horror came near enough."

"The bridges built in the last five or six years are perfectly safe, unless two trains should meet or a train run off the edge. Both of these accidents are extremely improbable. The railroad companies allow no iron bridges, improperly constructed, to remain. There are, to be sure—or so I have heard—a good many unsafe bridges, probably hundreds of them. It will take time for these to be replaced by iron or steel ones. The great danger with wooden bridges is from cinders and sparks. These drop on the wood, char in a little, and weaken the structure until an unusually heavy train or sudden jar causes a crash. There have been hundreds of accidents from this cause. So the wooden bridges must go. An iron bridge costs little more.

We've always been ahead of the world in bridge building, and we intend to stay there."

The following extract from the "Delaware Bridge Company's Album" bears upon what I have already said concerning bridge designing in America being a specialty.

"The construction of wrought iron bridges has attained such development within the past 10 years as to be now recognized as a separate and important branch of constructive engineering, essentially depending for its success upon the skill, experience and integrity of the engineer, who has specially devoted himself to the study and practice of the subject.

Good iron bridges are occasionally built by engineers in general practice, and there are still a few railroad companies which have a bridge construction department; but as a rule, bridges are built to-day by men who have endeavored to acquaint themselves with the intricate questions involved in the application of the general theory of skeleton structures to practice, and who have found that the subject was capable of a sufficient development to absorb their exclusive attention. In other words, bridges are built to-day by bridge-builders, and to become a bridge-builder demands such an amount of technical knowledge, coupled with, and partly the result of, large experience in design, and in the manipulation of materials, as will ensure the erection of structures which are not only scientifically sound in principle, but at the same time economical and durable."

The following from an article in the Chicago *Railroad Gazette* of July 1870 upon "English and American Iron Bridges" will also confirm some of my statements.

"Some two months ago tenders were solicited for the construction of iron railway bridges of spans of 100 and 200 feet, by the Intercolonial Railway of Canada, connecting Quebec and Halifax. This call was very generally responded to, there being tenders put in by nineteen English, one Belgian, and sixteen American bridge-builders.

The specification, which was a rigid one, called for uniformity of strength, but left the design open to each person. The bridges were all to be of wrought iron, capable of bearing 1¼ gross tons per lineal foot, in addition to their own weight, without straining the iron in tension to over 10,000 pounds per square inch. The iron of the 200 feet spans was to be capable of bearing 60,000 pounds per square inch before breaking, and that of the 100 feet spans 50,000 pounds per square inch.

Much interest was felt as to the result of this competition, which was virtually

one between English and American systems of bridge building. The decision was that the long spans were awarded to an American firm, Messrs. CLARKE, REEVES & CO., of Phœnixville, Pa., and the short spans to English bridge-builders, the Fairbairn Manufacturing Company, of Manchester. Of the thirty-six plans submitted, only three or four were rejected on account of not coming up to special strength.

The bridges of Clarke, Reeves & Co. were selected for the long spans, not only as being undoubtedly first-class, both in material and workmanship, but also as being the lowest responsible tender. Some curiosity has been expressed to know how American bridge-builders, using high-priced iron, and paying higher wages for labor than their English competitors, could yet build a less costly bridge.

While it is to some extent true that the specifications allowed of a lower quality and less expensive iron for the 100 than for the 200 feet span, yet one of the principal reasons why an American firm was lowest on the long and an English firm on the short spans is owing to the less weight of iron required by the American system of bridge, and this is more apparent the longer the span.

Some persons erroneously suppose that the more iron there is in a bridge the stronger it will be. But a little reflection will show that it is only the iron that is working, or, in other words, that is actually strained by the load, that contributes to the strength of the structure. All the rest is dead weight, and merely weighs down the bridge. In very short spans this is not disadvantageous, as it tends to diminish vibration, but in long spans where the weight of the bridge much exceeds that of the load passing over it, every pound of iron that does not contribute to the strength of the bridge is a positive injury. To illustrate this more clearly: if one bridge weighs 125 tons and another 250, and both are strained by the rolling load 10,000 pounds per square inch, the lighter is the stronger of the two. But if the 125 ton bridge be strained 10,000 pounds per square inch, while the 250 ton bridge is strained only 5000 pounds per square inch, then tne latter has really double the strength and double the life of the former; for half the iron may corrode away, and then the working area of the bar will be equal. It is not clearly perceiving this fact—that the strength of the bridge depends upon the working area of each part—that has led our English friends to make such heavy bridges.

In several plans, if the strains per square inch are alike for similar loads they must all be of the same strength, providing the connections are equally perfect. Some take more iron than others to effect the result, but the result is the same.

The lightness of American bridges is due—1st, to the concentration of material along the lines of strain, which enabled a lighter web system to be used, and hence a higher truss; 2d, to this greater height of truss, which throws less leverage on the upper and lower chord system, and hence requires less iron in their members; 3d, to the use of eye and pin connections instead of rivets, by which there is no waste of metal to compensate for the deduction of rivet-holes.

American bridges are stiffer vertically and better braced laterally than English bridges, their greater height giving less deflection under a load, and allowing of overhead bracing as well as that below the track.

But the less quantity of iron required to do the work is not the whole explanation of the less cost of American as compared with English bridges. A second and equally important reason is the less amount of manual labor required to construct and erect them—owing to the general use of machinery in forming all the parts.

English bridges are made of low-price iron and require a great deal of it, and a great deal of hand-labor in constructing and erecting.

American bridges have all their principal parts formed by machinery. They are of exact uniform dimensions, in similar spans, and hence perfectly interchangeable, like the parts of the locks of the American rifles, or of sewing-machines. Hence machine-labor can be applied to their manufacture, and the cost at the works reduced to a minimum.

But American bridges have still another advantage. They are so made that nearly all the work is done at the shops, and they can be erected with the least possible amount of labor, and that unskilled. In fact, the cost of erecting the staging is the principal expense; after that a 200 feet span can be erected and made self-sustaining in the space of two days, if necessary.

But the English bridge is only about half done when the scaffolding is built and the iron placed upon it. It has then to be riveted together, which is expensive, as the conveniences for such work at the site of a bridge are not often great. It is slow and tedious, requiring from two to three weeks to put together a 200 feet span.

Taking all these things into account, it will be seen how American bridge-builders have been able to compete with English firms on the large bridge at Buffalo, and in the recent case of the long span bridges of the Intercolonial Railroad of Canada."

I have dwelt at length upon the comparison of American and European bridges, for the reason that the Japanese railroad bridges are of the latter type.

It will now be necessary for me to criticize the railroad bridges of this country, and I hope you will excuse me for so doing. I have little hesitation in expressing my opinion thereon, knowing that the designs are not yours, but are the work of some of the present and former foreign employees of the Railway Department.

The first grave error to which I would call your attention is that both for economical and prudential reasons the spans are too short, the superior limit being one hundred feet. For any locality that bridge is the most economical, for which the total cost of both superstructure and foundations is a minimum, provided that the waterway be not so lessened as to endanger the structure from washout or to raise the flood level of the river enough to injure the surrounding country.

Now as the cost of foundations is always very uncertain and in most cases exceeds the estimate, it is clear that long spans, especially over the deepest part of the river, are liable to be more economical than short ones, and no one will deny that a span of one hundred feet is a short one. Again, for such spans the contraction of waterway is at least ten per cent, which, in addition to the piling up of the water by the impact of the current against the piers, will cause a decided increase in the flood level.

If an American engineer were sent to inspect and pass judgment upon a

Japanese railroad truss bridge, he would condemn it before getting within a hundred-yards of the structure, for all such bridges have pony trusses without any side bracing. This is objectionable for two very important reasons: first there is nothing to resist the wind pressure upon the top chord, and to prevent its overturning the truss; and second, when the top chord is not held laterally at the panel points or other places, its true length as a column must be about equal to the total length of span, when considered in respect to lateral deflection under load. It is quite evident that no pony trusses in this country have their top chords proportioned for the number of diameters found by dividing the length of span by the width of top chord plate.

As the before mentioned inspector would approach the bridge he would be struck, in fact horrified, by the absolute lack of lateral bracing; for one cannot imagine that the rivetting of the floor beams to the lower chords by four rivets at each end can give any lateral strength to the bridge when subjected to wind pressure. *There is just as much reason in this arrangement as there would be in omitting the diagonals from the trusses and rivetting the vertical posts to the outside of the top and bottom chords.* Such an arrangement might sustain a small balanced load, but an unevenly distributed load would certainly destroy the structure. The Japanese truss bridges are therefore wholly unfitted to resist the stresses produced by a whirlwind.

The next thing that would catch the inspector's eye would be the inclined struts of the Warren girder, formed by trussing, in the most inefficient manner possible, two very thin, wide bars. Such struts were experimented upon years ago in America and were unhesitatingly condemned. It needs no experiment, though, to show their inefficiency; for theory teaches that the strength of a strut increases with the radius of gyration of its section in respect to the neutral axis of that section; and it is evident to all that a flat bar has a very small radius of gyration.

The next parts that the inspector would notice would be the chords. In the upper there is a waste of material at all points except the centre; and the box form of the lower would condemn it immediately in his eyes. Concerning this point let me give you the opinion of A. P. Boller, Esq. C. E., a well known American engineer of acknowledged ability, as expressed in his treatise on "Iron Highway Bridges."

"In continuous box-shaped chords, the pin holes must be reinforced with thickening plates, not only to increase pin-bearing, but also to distribute the pressure delivered to the chord at each panel point over as much surface as possible. Further it is advisable that the increased sectional area required at each panel point, in approaching the centre, be placed in the *sides* of the box. as it is through the sides that the pin passes. It is not one of the least excellencies of the pin-connection system that the chords, posts and tension-members may be made to unite at the centre of their several sections, and by proportioning the box chord as above this may be accomplished very fully................This principle is about as far lost sight of in rivetted work as it is possible to be. In such work the chords have no stiffening along the inner edges of the vertical plates or sides to which the web system is rivetted, and the increase of area is made by rivetting on plates to the *upper* side of the top chord, or *lower* side of the bottom. The centre of section is not at the middle of the sides, as usually assumed, but approaches the top or bottom plates, and in large

spans, where the strains are great, necessitating a large area of section (placed mostly in the above plates) the centre of section approaches the plates very rapidly.

The rivetted system has of necessity, so many inperfections of design, of workmanship and material, in contrast with the above [pin-connected], *that, to obtain anything approaching equal strength on the same specification, it should only be used with a higher factor of safety.* It is probable that this difference is not less than 20 per cent: so that when a pin bridge is called for, having a factor of *five*, a rivetted bridge cannot be considered as approaching the same strength unless it is proportioned with a factor of *six*. The fact that a rivetted bridge is stiff or that its deflections may be small under a test, is no evidence of strength, which last depends upon other considerations than those applying to stiffness"

These remarks of Mr. Boller's are intended for lattice bridges, in which the web members are rivetted to the chords, but they are most of them applicable to the lower chords of the Japanese bridges, which are made continuous from end to end of span by rivetting. The Japanese truss bridges, although Warren girders, are not what may be properly termed pin-connected bridges, for the office of the lower chord pins is merely to transfer the web stresses to the chords.

The inspector would next turn his attention to details and would notice the apologies for stay plates containing one rivet at each end and spaced about three feet apart, which connect the opposite flanges on the under sides of the top chords; the heads on the main diagonals formed by rivetting a piece of plate on each side of the bar at the eye; and the smallness of the connecting plates and the pancity of rivets at the joints in the chords.

There is one thing that he would be sure not to overlook, and that is the absence of a guard rail or any arrangement to prevent a derailed car or locomotive from going through the bridge. This is indeed a grave fault, for derailed cars have been known to go long distances before the accident has been found out: the reason that no Japanese bridge has as yet been destroyed in this way is probably due to the excellent quality of the road bed and to the absence of severe frost.

The trouble with most English bridges and consequently with those of this country is that they are designed by railroad engineers, who have not made a special study of bridge designing, and are therefore incompetent to do the work entrusted to them.

Although I have made many inquiries I have been unable to ascertain anything certain concerning the designing of the Japanese bridges, in respect to either the assumed loads or the intensities of working stress, the invariable answer to my questions being that "the bridges are made according to designs prepared by foreigners." One engineer did tell me that the assumed live load for all cases was one ton (2240 pounds) per lineal foot. If such be the case, the short spans are too weak.

Thanks to the courtesy of Mr. Takanobu Kōno. M. E. and Mr. Yoshimura of the Kōbu Daigaku, I have been able to obtain the actual weights of iron in a number of the Japanese bridges. Of these I have chosen the following for the purpose of comparison with the bridges designed for this treatise.

A single track lattice girder of sixty-seven feet span on the Kōbe and Osaka line

weighs 31.5 long tons or 70,560 pounds. Another single track lattice girder of ninety-four feet span between Kioto and Osaka weighs 47 long tons or 105,280 pounds. A double track truss bridge of one hundred feet span weighs 77.6 long tons or 173,824 pounds.

By interpolating in Table I we find the weight of iron per lineal foot for a 67′ span to be about 800 pounds, and that for a 94′ span 852 pounds, making the total weights for these cases respectively 53,600 and 80,088 pounds.

A special calculation gave the total weight of iron for a 100′ span double track bridge as 175,750 pounds.

Now as the bridges of this treatise are provided with iron stringers and guard rails and oak ties, while the Japanese bridges have only wooden stringers it is evident that the former are at any rate the more economical; and, I think, that if you will take the trouble to carefully read the following chapters, you will conclude that they are also much better designed.

The reason why the double track bridge that I designed is proportionately so much heavier than the single track bridges is that the overhead bracing for reasons, which will appear further on, is necessarily very heavy.

But to return to the subject of American railroad bridges; I do not wish you to imagine that I consider them all perfect and in every way superior to the European. Unfortunately such is not the case, for many existing bridges in the United States are the work of inferior bridge companies and engineers, who have failed to pay proper attention to detail. Then again the bridges of twenty years ago are not heavy enough for the rolling loads of to-day, and moreover the science of bridge designing has made great progress in the last twenty years. But the lately erected bridges of the better class of American bridge companies are undoubtedly good, and it is with these in view that I have prepared this treatise, endeavouring in every respect not only to equal them in excellence of design but to improve upon them wherever I saw the opportunity. The styles of truss adopted are those of the Pratt and Whipple systems, or the single and double quadrangular trusses. That these forms are both the best and most economical is proved by their being almost universally adopted by the leading bridge builders of the United States; besides, I have shown in a paper entitled "Economy in Struts and Ties," by a method entirely practical, that vertical posts and inclined ties are more economical than any other arrangement; and these are the essential features of the Pratt and Whipple trusses.

You will notice that double track bridges and deck bridges have not been as fully treated as through and pony truss bridges: deck bridges are applicable to only high grade crossings, few of which will be found necessary in this country; while the double track bridges will not be needed, in all probability, for the next twenty years, by which time steel will have replaced iron in bridge construction. Nevertheless you will find that both these styles of structure have received sufficient attention to enable an engineer to design them with ease, the only difference being that he will have no diagrams similar to those on Plates XIV—XLII to guide him. In reference to these diagrams I would state that the dead loads and wind pressures had to be first assumed then checked, so that they do not agree exactly with those given in the

body of the work, the differences, however, are all within the limits allowed in good practice. Although the principles employed in designing plate girder spans have been fully elucidated, no examples have been worked out or diagrams given, for the reason that time and space do not permit; and there is no necessity therefor, because you have many examples of existing plate girder spans, which do not differ fundamentally from those which would be designed by the methods of this book. Nevertheless you will find that the latter will exceed the former in weight and efficiency.

No special treatment has been given for skew bridges, for none is needed; the methods for designing them being precisely the same as those for designing other bridges. Whenever it is convenient to do so the panel length of a skew bridge should be chosen so that the shoe of one truss comes opposite the first panel point of the other truss, in order that the floor beams may be at right angles to the planes of the trusses, both for economical reasons and to avoid using single beam hangers. This arrangement can often be made by shortening the panel length a little, and, if it be allowable, slightly changing the angle of the skew. Even if it be impracticable to make this arrangement, it is usually better in skew bridges to advance the ends of the floor beams at one side of the bridge by one or even two panel lengths, if by so doing the floor beams be shortened.

The ton used in the following chapters is the American or short ton of two thousand pounds: it will be found much more convenient than the long ton.

It seems almost unnecessary to state, except for the benefit of foreign readers, that the gauge of track is 8′ 6″, that the distance between centre lines of inner rails, where there is a double track, is 6′ 2½″, and that the width of the head of a rail is 2½″, making the distance between centres of parallel tracks 9′ 10½″.

In making the designs on Plates XIV—XLII American iron was employed, the reason being, as can be seen from the next chapter, that the European channel sections do not have the necessary range in weight and are with a single exception, limited to a depth of twelve inches. Carnegie's sections have been exclusively used, because not only does his company roll more iron than any other company in the United States, but they are also tabulated in a more convenient form than are any other sections. On this account I have copied from his "Pocket Companion," with the kind permission of the Company, all the tables that I think will be of any use to you.

It would be well for each engineer in this country, who has anything to do with ironwork, to provide himself with the books of sections of all the manufacturing companies mentioned in Chapter II.

Concerning the cost per pound of finished bridges of the American type I have made many inquiries both in England and America. The American price is at present about 4½ cents f. o. b.: the English manufacturers with but one exception refuse to quote prices without seeing working drawings. This exceptional company makes the price about half a cent less or 4 cents gold. The freight charges from England and America differ only a few cents per ton.

As to where it is better to have bridges manufactured each engineer must judge for himself. My opinion is that for plate girder spans it would be more

economical and satisfactory to import the iron and manufacture them in this country; that for all single track bridges and all ordinary double track bridges it would be cheaper and better to have them manufactured in America, and that for all double track bridges which are so heavy as to necessitate the use of built channels for the top chords and batter braces it would pay to purchase them in England.

The extra half cent per pound may appear to some of you so great as to nullify the economy of purchasing bridges in America, but you will find that such for various reasons is not the case.

First the want of the proper channel sections will generally necessitate the use of built channels for the top chords and batter braces, as in the present Japanese bridges, and sometimes even for posts. By reason of the shopwork thereon such channels are more expensive than rolled sections; nor are they as strong or of as pleasing an appearance. Secondly bridges of the American style can be built much better in America than in England, where not only are the workmen unused to this style of work, but also the manufacturers are unprovided with the necessary special appliances.

Thirdly such bridges can be built much more quickly in America; and time is money. It was only a few weeks ago that I read in an American technical journal that one of the South American countries had decided to purchase all its iron bridges in the United States rather than in England, simply on account of the delay.

And finally there may not be such a difference as half a cent per pound, because the prices of iron and labour are falling in America, and it was only one company in England that was willing to quote any prices at all. *

Whenever there is a large piece of ironwork to be done in Japan, it would be well, if time permit, to send to the principal English and American manufacturers for prices. In this way only can it be determined to whom it will be best to award the contract.

The addresses of the principal manufacturers can be found among the advertisements in the London *Engineering* and in the New York *Engineering News and American Contract Jurnal.* It would be out of place for me to recommend to you in this work any particular manufacturers, though I have no objection to giving to any of you individually my opinion as to what shops in America do the best work.

---

* Since the M. S. of this book was sent to the printer, I have heard from the engineer of one of the largest and best known bridge companies of the United States, that American iron manufacturers are underselling the English by three or four pounds sterling per ton; and that American iron is about to be used in Australia. The same engineer writes as follows. "Our structural iron is notoriously better than English.......Besides the lower price and better quality of metal in American work, the latter has the advantage of far greater facility in erection. English work requires skilled labor to put together in the field, on account of its rivetting. Our structures can be put up in Japan by the natives themselves. We can ship work to Japan with erection plans, where it can be received and put up by the Japanese themselves. We are constantly doing this for South American parties."

You can depend upon getting good work done by the principal ones, but may be disappointed if you go to some of the less important manufacturers.

Those of you, who have read my treatise on "The Designing of Ordinary Iron Highway Bridges," will notice that some portions of the same are copied *verbatim* into this work: this needs no apology, for highway and railroad bridges have many features in common. This fact will account for the two different styles of spelling, which you will probably notice: mine is the English method, but my book having been printed in the United States was there translated into American: it would take more time than it is worth to make all the corrections needed for uniformity.

The references to Chapters instead of pages has been a necessity, as this book is not to be electrotyped. Any inconvenience resulting therefrom can be avoided by referring to the Index, which, on this account has been made very complete.

Before closing this chapter I wish to acknowledge with many thanks my indebtedness to Theodore Cooper, Esq. C. E. for permission to use a part of his valuable specifications, and to my assistants Messrs. T. Fukuda and Y. Nakajima for aid in preparing plates and tables. Without Mr. Nakajima's skilled assistance I would have been unable to undertake such an amount of labour as the preparation of this treatise has involved. Acknowledgements of assistance from other engineers are made in various places in the remaining chapters.

With many apologies for having detained you so long upon this introductory letter,

I am, gentlemen,
very respectfully yours,
J. A. L. Waddell.

Tokio. Jan. 1885.

CHAPTER II.

SECTIONS OF IRON.

The larger part of the data contained in this chapter was obtained through the courtesy of Jas. Forrest, Esq, C. E, Secretary of the Institution of Civil Engineers, London, who very kindly complied with the author's request to collect for him books of sections and tabulated dimensions of all the bridge iron used in Great Britain. Messrs Carnegie Bros. and Co. of Pittsburgh Pa. have been so kind as to furnish most of the remaining data.

Although this collection of sections of bridge iron is not complete, yet it contains the dimensions of nearly all the sections used in Great Britain and America.

Owing to want of dimensions the shapes rolled by the following British manufacturers have not been recorded; Wm. Whitwell and Co, Stockton-on-Tees; Jos. Whitman and Sons, Perseverance Ironworks, Leeds; Dorman, Long and Co, Middlesbrough; and the Skerne Ironworks Company, Darlington; but judging by the illustrations in Mr. Edward Hutchison's "Girder-Making and the practice of Bridge Building in Wrought Iron" one may conclude that these firms roll no shapes very different from those here recorded.

In America there are other manufacturers than the three here given, but these three roll most of the bridge iron used in that country.

The tables of sections have been made as complete as the data at the author's disposal would allow, so if any manufacturer's produce has been slighted or that of any other received undue prominence, it is to be hoped that the author will be pardoned.

Sections of Iron manufactured by the Butterley Co.
Silverdale, North Staffordshire.

ANGLE IRON.

Size in Inches.	Thickness in Inches.	Weight per foot.	Area in Square. Inches.
		lbs.	
7 × 7	¾ to 1¼	33.12 to 53.12	9.94 to 15.96
10 × 3½	7/16 „ ⅝	19.02 „ 26.82	5.71 „ 8.05
9 × „	„ „ „	17.59 „ 24.74	5.28 „ 7.42
8 × 4½	„ „ „	17.59 „ 24.74	5.28 „ 7.42
6 × 6	½ „ 1	19.17 „ 36.67	5.75 „ 11.00
8 × 3½	⅜ „ ¾	13.91 „ 26.87	4.17 „ 8.06
5½ × 5½	½ „ ¾	17.50 „ 25.62	5.25 „ 7.69
7 × 3½	7/16 „ ⅝	14.67 „ 20.57	4.40 „ 6.17
6½ × 4	½ „ ⅝	16.67 „ 20.57	5.00 „ 6.17
6 × 4	„ „ „	15.83 „ 19.53	4.78 „ 5.86
5 × 5	„ „ ¾	15.83 „ 23.12	4.78 „ 6.94
7 × 3	⅜ „ „	12.03 „ 23.12	3.61 „ 6.94
6 × 3½	„ „ „	11.41 „ 21.87	3.42 „ 6.56
4½ × 4½	„ „ „	10.78 „ 20.62	3.23 „ 6.19
6 × 3	„ „ „	10.78 „ 20.62	3.23 „ 6.19
4¼ × 4¼	„ „ „	10.16 „ 19.37	3.05 „ 5.81
5½ × 3	„ „ „	10.16 „ 19.37	3.05 „ 5.81
5 × 3½	„ „ „	10.16 „ 19.37	3.05 „ 5.81
4¾ × 3¾	„ „ „	10.16 „ 19.37	3.05 „ 5.81
4 × 4	„ „ „	9.53 „ 18.12	2.86 „ 5.44
5 × 3	„ „ „	9.53 „ 18.12	2.86 „ 5.44
4½ × 3	„ „ „	8.91 „ 16.87	2.67 „ 5.06
4 × 3½	„ „ „	8.91 „ 16.87	2.67 „ 5.06
3½ × 3½	„ „ „	8.28 „ 15.62	2.48 „ 4.69
4 × 3	„ „ „	8.28 „ 15.62	2.48 „ 4.69
4 × 3	5/16 „ „	6.97 „ 15.62	2.09 „ 4.69
3¼ × 3¼	„ „ ⅝	6.45 „ 12.24	1.94 „ 3.67
3½ × 3	„ „ „	6.45 „ 12.24	1.94 „ 3.67
4 × 2½	„ „ „	6.45 „ 12.24	1.94 „ 3.67
3 × 3	¼ „ „	4.79 „ 11.20	1.44 „ 3.36
4 × 2	5/16 „ ½	5.92 „ 9.17	1.78 „ 2.75
3½ × 2½	„ „ „	5.92 „ 9.17	1.78 „ 2.75
2¾ × 2¾	¼ „ „	4.37 „ 8.33	1.31 „ 2.50
3 × 2½	„ „ „	4.37 „ 8.33	1.31 „ 2.50
2½ × 2½	3/16 „ „	3.01 „ 7.50	.90 „ 2.25
3 × 2	„ „ „	3.01 „ 7.50	.90 „ 2.25
2¼ × 2¼	„ „ „	2.70 „ 6.67	.81 „ 2.00
2½ × 2	„ „ „	2.70 „ 6.67	.81 „ 2.00
2 × 2	„ „ „	2.38 „ 5.83	.71 „ 1.75

CHANNEL IRON.

Depth in Inches.	Weight per foot in lbs.	Area of Section.	Thickness of Web in Inches.	Width of Flange.
12	40 to 42	12.00 to 12.60	11/16	3¾
10	28 „ 30	8.40 „ 9.00	½	3½
8	26 „ 28	7.80 „ 8.40	„	4
7	21 „ 23	6.30 „ 6.90	⅜	2
6	15 „ 14	3.90 „ 4.20	⅜	2½
6	11 „ 12	3.30 „ 3.60	5/16	2½
5	18 „ 19	5.40 „ 5.70	½	3½
4½	11 „ 12	3.30 „ 3.60	„	1½
4½	6 „ 6½	1.80 „ 1.95	5/16	¾
4	5¾ „ 6	1.73 „ 1.80	„	¾

Note! The above sections can be slightly increased in the thickness of the web, but will also be to the same extent *wider* in the *flanges*.

In all probability the Butterley Co. rolls also tee irons and I beams.

Sections of Iron manufactured by C. C. Dunkerkley, & Co. Manchester.

GIRDERS.

Section Number.	Section.	Weight. Maxm. per foot.	Weight. Minm. per foot.
	Inches.	lbs.	lbs.
D 30	20 × 7 × 7	100	90
D 33	18 × 6¾ × 6¾	80	77
D 34	17 × 6¼ × 6¼	77	70
D 32	16 × 6 × 6	68	57
D 35	15 × 5 × 5	67	50
D 82	14 × 6 × 6	64	54
D 85	13 × 7 × 7	65	65
D 36	12⅝ × 5¼ × 5¼	60	45
D 37	12 × 7⅜ × 7⅜	81	77½
D 27	12 × 6 × 6	67	52
D 25	12 × 5 × 5	55	39
D 86	10 × 6 × 6	56	56
D 23	10 × 5 × 5	47	35
D 21	10 × 4½ × 4½	41	30
D 40	9½ × 4½ × 4½	36	27
D 19	9¼ × 3¾ × 3¾	32	21½
D 43	9¼ × 3½ × 3½	28	21
D 47	9 × 6 × 6	40	40
D 93	9 × 4½ × 4½	32	32
D 44	9 × 4 × 4	30	23
D 45	8¾ × 2¾ × 2¾	27	16½
D 46	8 × 6 × 6	37	33
D 17	8 × 5 × 5	37	29
D 15	8 × 4 × 4	30	21
D 48	8 × 2½ × 2½	22	14
D 13	7⅛ × 3¾ × 3¾	24	18
D 94	7 × 6 × 6	36	35
D 49	7 × 4 × 4	24	19½
D 83	7 × 3 × 3	18	18
D 6	7 × 2¼ × 2¼	18	11
D 87	6½ × 3½ × 3½	21	15¾
D 9	6¼ × 3⅛ × 3⅛	21½	15½
D 22	6 × 5 × 5	32	25
D 11	6 × 3 × 3	18½	14½
D 2	6 × 2⅛ × 2⅛	16½	10
D 41	5½ × 2¾ × 2¾	15	12
D 42	5½ × 2 × 2	11½	8½
D 20	5 × 4½ × 4½	25	21
D 18	5 × 3 × 3	15½	12
D 7	4¾ × 2 × 2	12	7½
D 16	4 × 3 × 3	13½	11¾
D 4	4 × 1¾ × 1¾	9½	6½

CHANNELS.

Section Number.	Section.	Weight. Maxm. per foot.	Weight. Minm. per foot.
	Inches.	lbs.	lbs.
D 51	11¾ × 3 × 3	30	26
D 54	10¼ × 3½ × 3½	27	23
D 91	10 × 3⅛ × 3⅛	31	25½
D 57	9¾ × 3⅛ × 3⅛	27¾	21½
D 63	9⅜ × 3½ × 3½	31	22
D 60	9¼ × 3¼ × 3¼	30	21
D 62	8 × 3½ × 3½	24	22
D 61	7¼ × 2½ × 2½	17	13
D 65	6⅞ × 2⅝ × 2⅝	19	14
D 64	6¼ × 2½ × 2½	17	12
D 92	6 × 2⅝ × 2¾	16¼	12¾
D 59	5¾ × 2⅝ × 2⅝	13½	10½
D 58	4⅝ × 2½ × 2½	13½	11
D 56	4⅛ × 2½ × 2½	11½	9
D 55	4 × 2 × 2	13	9½

Rolls turned for new Sections.

GIRDERS KEPT IN STOCK.

Lengths up to 40 feet.

Number.	Sectional Dimensions. Web.	Sectional Dimensions. Top Flange.	Sectional Dimensions. Bottom Flange.	Approximate Weight per foot.
	Inches.	Inches.	Inches.	lbs.
D 30	20	7	7	100
D 32	16	6	6	62
D 82	14	6	6	56
D 27	12	6	6	56
D 25	12	5	5	43
D 23	10	5	5	40
D 21	10	4½	4½	33
D 19	9¼	3¾	3¾	25
D 17	8	5	5	30
D 15	8	4	4	24
D 13	7	3¾	3¾	20
D 6	7	2¼	2¼	14
D 22	6	5	5	29
D 11	6	3	3	16
D 2	6	2⅛	2⅛	11
D 20	5	4½	4½	23
D 18	5	3	3	13
D 7	4¾	1¾	1¾	8
D 16	4	3	3	11
D 4	4	1¾	1¾	7

The author is unable to state whether Messrs Dunkerkley & Co. roll tee and angle irons or whether they make a specialty of channels and I beams.

Sections of iron sold by Measures, Bros. & Co.,
Southwark St. London.

CHANNELS.

MINIMUM.			MAXIMUM.		
Sections.	**Thicks. of Web.**	**Wt. per ft.**	**Sections.**	**Thicks. of Web.**	**Wt. per ft.**
11 7/8″ × 3″	3/8″	25	11 7/8″ × 3 1/4″	1/2″	33 1/2
9 7/8″ × 3 1/16″	7/16″	23	9 7/8″ × 3 11/16″	9/16″	30 1/4
9 3/4″ × 3 5/16″	3/8″	23 1/2	9 3/4″ × 3 1/8″	1/2″	27 5/8
7 7/8″ × 3 1/8″	3/8″	19 1/2	8″ × 3 3/8″	15/32″	25
5 13/16″ × 2 3/8″	5/16″	10 1/2	5 3/4″ × 2 9/16″	13/32″	14
4 3/16″ × 2 5/16″	5/16″	9 3/4	4 1/8″ × 2 13/16	13/32″	11 3/4

GIRDERS.

Depth.	Width.	Wt. per ft.	Remarks.	Depth.	Width.	Wt. per ft.	Remarks.
16″	6″	62	In Stock.	7″	3 3/4″	20	In Stock.
14″	6″	60	„ „	6″	5″	29	„ „
12″	6″	56	„ „	6 1/4″	3 3/8″	16	„ „
12″	5″	42	„ „	5″	4 1/2″	23	„ „
10″	5″	36	„ „	4 7/8″	3″	13	„ „
10″	4 1/2″	32	„ „	4″	3″	12	„ „
9 1/2″	4 1/2″	29	„ „	7″	2 1/4″	14	Rounded Flan ges
9 1/4″	3 3/4″	24	„ „	6 1/4″	2″	11	„ „
8″	5″	29	„ „	4 3/4″	1 3/4″	8	„ „
8″	4″	22	„ „	4″	1 3/4″	7	„ „

Measures, Bros. & Co. roll also a number of angle and tee irons, the weights and dimensions of which are not given in their book of sections.

Sections of iron manufactured by Philip Williams and Sons, Wednesbury Oak Ironworks, Tipton, Staffordshire.

FLATS.

1½	inch wide,	× ¼″	to 1¼	inch thick	4¼	inch wide,	× ¼	to 1	inch thick
1⅝	„	× „	„	„	4½	„	× „	„	„
1⅝	„	× 3/16	„	„	4¾	„	× „	„	„
1¾	„	× „	1½	„	5	„	× „	„	„
1⅞	„	× „	„	„	5½	„	× „	„	„
2	„	× 3/16	1¾	„	6	„	× „	„	„
2⅛	„	× ¾	1	„	6¼	„	× ⅜	1¼	„
2¼	„	× „	2	„	6½	„	× ½	1¾	„
2⅜	„	× „	⅞	„	7	„	× „	1	„
2½	„	× „	2¼	„	7½	„	× „	1¾	„
2⅝	„	× „	2⅞	„	8	„	× „	2	„
2¾	„	× „	2½	„	8½	„	× „	2⅛	„
3	„	× „	„	„					
3¼	„	× „	2⅞	„					
3½	„	× „	2	„					
3¾	„	× „	„	„					
4	„	× „	„	„					

ROUNDS.

¾, 13/16, ⅞, 15/16, 1, 1 1/16, 1⅛, 1 3/16, 1¼, 1⅜, 1½, 1⅝, 1¾, 1⅞, 2, 2⅛, 2¼, 2⅜, 2½, 2⅝, 2¾, 2⅞, 3, 3⅛, and 3¼ inch.

SQUARES.

¾, 13/16, ⅞, 15/16, 1, 1 1/16, 1⅛, 1 3/16, 1¼, 1⅜, 1½, 1⅝, 1¾, 1⅞, 2, 2⅛, 2¼, 2⅜, 2½, 2⅝, 2¾, 2⅞, and 3 inch.

ANGLE IRON.

2 × ¼ to ⅜ in. 2½ × 5/16 to ½ in. 2¾ & 3 × ⅜ to ⅝ in. 3½ × ½ to ⅝ in. and 4 × ½ to ¾

SECTIONS OF IRON AND STEEL, *Manufactured by*
THE PATENT SHAFT AND AXLETREE CO., LIMITED, WEDNESBURY,
BRUNSWICK, OLD PARK, & MONWAY IRON & STEEL WORKS,

EQUAL ANGLES.

$2 \times 2 \times \frac{3}{16}$ to $\frac{7}{16}$
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{4}$ — $\frac{1}{2}$
$2\frac{1}{2} \times 2\frac{1}{2} \times$,, — ,,
$2\frac{3}{4} \times 2\frac{3}{4} \times$,, — $\frac{9}{16}$
$3 \times 3 \times$,, — $\frac{3}{4}$
$3\frac{1}{4} \times 3\frac{1}{4} \times \frac{1}{4}$ — $\frac{5}{8}$
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ — ,,
$3\frac{3}{4} \times 3\frac{3}{4} \times$,, — $\frac{5}{8}$
$4 \times 4 \times$,, — $\frac{7}{8}$
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{1}{2}$ — $\frac{7}{8}$
$5 \times 5 \times$,, — ,,
$6 \times 6 \times$,, — $\frac{3}{4}$
$6\frac{1}{8} \times 6\frac{1}{8} \times \frac{11}{16}$ — 1
$5\frac{1}{2} \times 5\frac{1}{2} \times \frac{1}{2}$ — $\frac{7}{8}$

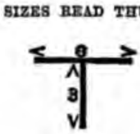
UNEQUAL ANGLES.

$2\frac{1}{4} \times 2 \times \frac{3}{16}$ — $\frac{7}{16}$
$2\frac{1}{2} \times 1\frac{3}{4} \times \frac{1}{4}$ — $\frac{3}{8}$
$2\frac{1}{2} \times 2 \times$,, — ,,
$2\frac{3}{4} \times 1\frac{1}{2} \times \frac{5}{16}$ — $\frac{1}{2}$
$2\frac{3}{4} \times 2 \times \frac{1}{4}$ — $\frac{7}{16}$
$3 \times 2 \times$,, — $\frac{3}{4}$
$3 \times 2\frac{1}{2} \times$,, — ,,
$3\frac{1}{8} \times 1\frac{1}{8} \times$,, — ,,
$3\frac{1}{2} \times 1\frac{3}{4} \times \frac{1}{4}$ — $\frac{3}{8}$
$3\frac{1}{2} \times 2\frac{1}{2} \times$,, — $\frac{3}{4}$
$3\frac{1}{2} \times 3 \times$,, — ,,
$4 \times 2 \times \frac{1}{4}$
$4 \times 2\frac{1}{2} \times \frac{3}{8}$ — $\frac{1}{2}$
$4 \times 3 \times$,, — $\frac{3}{4}$
$4 \times 3\frac{1}{2} \times$,, — $\frac{1}{2}$
$4\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ — $\frac{1}{2}$
$4\frac{1}{2} \times 3 \times \frac{3}{8}$ — $\frac{3}{4}$
$4\frac{1}{2} \times 3\frac{1}{2} \times$,, — $\frac{1}{2}$
$5 \times 3 \times$,, — $\frac{3}{4}$
$5 \times 3\frac{1}{2} \times$,, — $\frac{3}{4}$
$5 \times 4 \times$,, — $\frac{7}{8}$
$5\frac{1}{4} \times 1\frac{3}{4} \times \frac{1}{4}$
$6 \times 2\frac{1}{2} \times \frac{13}{32}$ — $\frac{5}{8}$
$6 \times 3 \times \frac{3}{8}$ — $\frac{3}{4}$
$6 \times 3\frac{1}{2} \times$,, — ,,
$6\frac{1}{4} \times 3\frac{3}{4} \times \frac{7}{16}$ — $\frac{9}{16}$
$6\frac{5}{8} \times 2 \times \frac{5}{16}$
$7\frac{1}{2} \times 4 \times \frac{7}{16}$
$8 \times 3 \times$,, — $\frac{5}{8}$
$8 \times 3\frac{1}{2} \times \frac{1}{2}$ — $\frac{5}{8}$
$8 \times 4 \times \frac{7}{16}$ — $\frac{1}{2}$
$8 \times 4\frac{1}{2} \times \frac{1}{2}$ — $\frac{5}{8}$
$9 \times 4 \times \frac{3}{8}$ — $\frac{1}{2}$
$9\frac{1}{4} \times 2\frac{1}{8} \times \frac{9}{16}$ — $\frac{5}{8}$
$11 \times 2\frac{5}{8} \times \frac{5}{8}$ — $\frac{1}{2}$
$11 \times 4 \times \frac{3}{8}$
$4\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{8}$
$8 \times 4\frac{1}{2} \times \frac{1}{2}$ — $\frac{7}{16}$
$6 \times 4\frac{1}{2} \times \frac{5}{8}$ — $\frac{3}{4}$
$3 \times 2\frac{3}{4} \times \frac{1}{4}$ — $\frac{3}{8}$
$6 \times 4 \times \frac{1}{2}$

TEE IRON.

$3 \times 1\frac{1}{2} \times \frac{3}{8}$ & $\frac{7}{16}$
$3 \times 1\frac{3}{4} \times \frac{3}{8}$
$3 \times 2 \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 2\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 3 \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 3\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 4 \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 5 \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 5\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$3 \times 6 \times \frac{3}{8}$
$3 \times 6 \times \frac{3}{8}$ & $\frac{1}{2}$
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$3\frac{1}{2} \times 3 \times \frac{7}{16}$, $\frac{1}{2}$, & $\frac{5}{8}$
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$3\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{8}$
$4 \times 2 \times \frac{3}{8}$ & $\frac{1}{2}$
$4 \times 2\frac{9}{16} \times \frac{1}{2}$
$4 \times 2\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$4\frac{1}{2} \times 2 \times \frac{3}{8}$
$4\frac{1}{2} \times 2 \times \frac{1}{2}$
$4 \times 3 \times \frac{3}{8}$ & $\frac{1}{2}$
$4 \times 3\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$4 \times 4 \times \frac{3}{8}$ & $\frac{1}{2}$
$4 \times 6 \times \frac{1}{2}$
$4\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$5 \times 2\frac{1}{2} \times \frac{3}{8}$, $\frac{7}{16}$, & $\frac{1}{2}$
$5 \times 3 \times \frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, & $\frac{5}{8}$
$5 \times 3\frac{1}{2} \times \frac{3}{8}$ & $\frac{1}{2}$
$5 \times 4 \times \frac{5}{8}$
$5 \times 6 \times \frac{9}{16}$ or $\frac{5}{8}$
$5 \times 5 \times \frac{1}{2}$ & $\frac{5}{8}$
$5 \times 4\frac{1}{2} \times \frac{5}{8}$ & $1\frac{1}{8}$
$5\frac{1}{2} \times 3 \times \frac{1}{2}$ or $\frac{11}{16}$
$6 \times 3 \times \frac{3}{8}$
$6 \times 3 \times \frac{1}{2}$, $\frac{9}{16}$, & $\frac{5}{8}$
$6 \times 3\frac{1}{4} \times \frac{3}{8} \times \frac{3}{4}$
$6 \times 3\frac{1}{2} \times \frac{1}{2}$
$6 \times 3\frac{1}{2} \times \frac{5}{8} \times \frac{1}{2}$
$6 \times 4 \times \frac{1}{2} \times \frac{3}{4}$
$6 \times 4 \times \frac{1}{2}$, $\frac{5}{8}$, & $\frac{3}{4}$
$6\frac{3}{4} \times 3\frac{3}{4} \times \frac{3}{4} \times \frac{5}{8}$
$6\frac{3}{4} \times 5\frac{1}{8} \times \frac{7}{8} \times 1\frac{3}{4}$
$6\frac{5}{8} \times 7 \times 1\frac{1}{16} \times 2$
$9\frac{1}{8} \times 4 \times 1 \times 3\frac{1}{4}$
$4 \times 4\frac{1}{2} \times \frac{1}{2}$
$6\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2} \times \frac{5}{8}$
$5 \times 3 \times \frac{1}{4} \times \frac{5}{16}$

SIZES READ THUS,



GIRDER IRON.

$4\frac{3}{4} \times 4 \times 2\frac{3}{4} \times \frac{3}{8}$
$7 \times 4\frac{1}{2} \times 4 \times \frac{1}{2}$
$7 \times 6 \times 6 \times \frac{1}{2}$
$7\frac{1}{2} \times 4\frac{3}{4} \times 4\frac{3}{4} \times \frac{1}{2}$
$8 \times 4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{8}$
$8 \times 5 \times 5 \times \frac{5}{8}$
$8\frac{1}{4} \times 3 \times 3 \times \frac{3}{8}$
$9 \times 7 \times 7 \times \frac{3}{4}$
$9\frac{1}{4} \times 3\frac{11}{16} \times 3\frac{11}{16} \times \frac{5}{8}$
$9\frac{1}{4} \times 3\frac{13}{16} \times 3\frac{13}{16} \times \frac{1}{2}$
$10\frac{1}{4} \times 6\frac{1}{4} \times 6\frac{1}{4} \times 1\frac{1}{8}$

CHANNEL IRON.

$4 \times 2\frac{1}{4} \times \frac{1}{2}$
$4 \times 2\frac{1}{2} \times 1 \times \frac{3}{8}$
$4\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{8}$
$5 \times 2\frac{3}{4} \times \frac{3}{8}$
$6 \times 2\frac{1}{2} \times \frac{1}{2}$
$6 \times 3\frac{1}{2} \times \frac{1}{2}$
$6 \times 4 \times \frac{19}{32}$
$7 \times 2\frac{3}{4} \times \frac{3}{8}$
$7 \times 3\frac{1}{2} \times \frac{1}{2}$
$7 \times 4 \times 1\frac{1}{32}$
$7\frac{7}{8} \times 2\frac{1}{2} \times \frac{5}{16}$
$7\frac{7}{8} \times 3\frac{1}{8} \times \frac{3}{8}$
$8 \times 3\frac{1}{2} \times \frac{1}{2}$
$8 \times 3\frac{5}{8} \times 1\frac{1}{8}$
$8\frac{1}{4} \times 2\frac{1}{2} \times \frac{5}{8}$
$9 \times 3 \times \frac{3}{8}$
$9 \times 3\frac{1}{2} \times \frac{5}{8}$
$9 \times 4 \times \frac{3}{8}$
$9 \times 4\frac{1}{2} \times \frac{11}{32}$
$9 \times 4\frac{1}{2} \times \frac{1}{2}$
$9\frac{1}{4} \times 3\frac{7}{8} \times \frac{11}{16}$
$9\frac{1}{4} \times 3\frac{1}{2} \times \frac{3}{8}$
$10 \times 3\frac{1}{4} \times \frac{3}{8} \times \frac{5}{16}$
$10\frac{1}{2} \times 3\frac{1}{4} \times 2\frac{1}{2} \times \frac{11}{32}$
$12 \times 3 \times 3 \times \frac{5}{8}$

CHANNEL SECTIONS MAY BE ROLLED.

$\frac{1}{32}$ to $\frac{3}{16}$ thicker.

ROUNDS,

$\frac{1}{4}$ to $6\frac{1}{2}$-in. diam.

SQUARES,

$\frac{1}{4}$ to 5-in.

FLATS,

$\frac{5}{8}$ to 13 $\frac{3}{8}$-in. wide.

Sections & Sizes of Rolled Iron Manufactured by the Shelton Bar Iron Company, Stoke on Trent, Staffordshire and 122, Cannon St. London E.C.

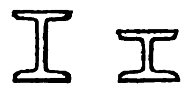
CHANNELS.

Depth in Inches.	Width of flange in inches.		Thickness of Web in inches.		Weight per foot in lbs.	
	Min.	Max.	Min.	Max.	Min.	Max.
4	2	2 1/4	1/4	3/8	6.75	10.08
4	2	2 1/8	1/2	5/8	11.50	14.00
5	1 1/2	1 3/4	3/8	5/8	9.10	13.25
5	2 7/16	2 3/8	5/16	1/2	10.50	13.48
5	2 3/4	3	3/8	5/8	12.00	18.00
6	2 7/16	2 3/4	5/16	5/8	12.75	18.00
7	2 11/16	2 7/8	5/16	1/2	14.00	18.00
7 7/8	2 7/16	2 9/16	5/16	7/16	14.00	18.00
7 7/8	2 11/16	2 15/16	5/16	13/32	16.00	24.00
7 7/8	3 1/8	3 5/16	3/8	13/32	18.00	23.00
8 1/4	2 7/16	2 5/8	5/16	1/2	14.50	20.75
8 15/16	2 15/16	3 1/16	5/16	7/16	17.00	21.00
9 1/16	2 7/8	3 1/8	3/8	5/8	20.00	26.50
9 1/4	3 1/4	3 7/16	3/8	9/16	21.50	26.00
9 1/4	3 1/2	3 11/16	3/8	9/16	22.00	30.00
10	3 7/16	3 5/8	7/16	5/8	24.00	28.00
12	3 13/16	3 15/16	9/16	11/16	37.50	47.00

ANGLES.

Size.	Thickness in inches.		Weight per foot in lbs.		Size.	Thickness in inches.		Weight per foot in lbs.	
	Min.	Max.	Min.	Max.		Min.	Max.	Min.	Max,
″ ″	″	″			″ ″	″	″		
2 × 2	3/16	7/16	2.38	5.20	4½ × 3½	⅜	¾	9.53	18.13
2¼ × 1¾	¼	½	3.13	5.83	4½ × 4½	⅜	⅞	10.78	23.70
2¼ × 2¼	3/16	9/16	2.70	7.38	5 × 3	⅜	⅞	9.53	20.78
2½ × 1½	¼	½	2.92	5.42	5 × 3½	⅜	¾	10.16	19.38
2½ × 2	¼	⅜	2.70	5.16	5 × 4	⅜	¾	10.78	20.63
2½ × 2½	¼	⅝	3.96	9.11	5 × 4½	⅜	¾	11.41	21.88
2¾ × 2¾	¼	⅝	4.38	10.16	5 × 5	½	1	15.83	30.00
3 × 2	¼	½	3.96	7.50	5½ × 3½	7/16	¾	12.49	20.63
3 × 2½	¼	¾	4.38	11.88	6 × 2½	5/16	⅞	8.53	22.24
3 × 3	¼	¾	4.77	13.13	6 × 3	⅜	¾	10.78	20.63
3 × 3	⅜	¾	7.03	13.13	6 × 3½	⅜	1	11.41	28.33
3 × 3	½	1	9.17	19.20	6 × 4	⅜	1	12.03	36.00
3¼ × 3¼	¼	¾	5.21	14.38	6 × 5	½	1	17.50	35.33
3½ × 1⅞	⅜	⅝	6.25	9.90	6 × 6	½	1	19.17	36.67
3½ × 2½	¼	11/16	4.79	12.17	7 × 3	7/16	⅞	13.95	26.61
3½ × 3	5/16	¾	6.45	14.38	7 × 3½	7/16	⅞	14.67	28.07
3½ × 3½	¼	⅞	5.63	17.86	8 × 3	⅜	⅞	13.28	29.53
4 × 2	¼	¾	4.79	13.13	8 × 3½	7/16	⅞	16.13	30.99
4 × 2½	5/16	½	6.45	10.00	8 × 4	7/16	⅞	16.86	32.45
4 × 3	¼	¾	5.63	15.63	8 × 4½	½	1	20.00	38.33
4 × 3½	⅜	1	8.91	21.67	9 × 3½	7/16	⅞	17.59	33.91
4 × 4	¼	⅞	6.46	20.78	10 × 3½	½	⅞	21.67	36.82
4½ × 3	⅜	¾	8.91	16.88					

JOISTS.



"	"	"			lbs. per foot.		
5 ×	3 ×	3/8	and up		10½	to	14
5½ ×	2 ×	5/16	,,		9	,,	15
6 ×	3 ×	3/8	,,		15	,,	19
6½ ×	3 1/16 ×	3/8	,,	"	17½	,,	20
8¼ ×	3 ×	5/16	to	7/16		from	17½
8¼ ×	3 ×	7/16	,,	9/16		,,	20½
9½ ×	3⅜ ×	3/8	and up		22½	to	28
9¼ ×	3 1/16 ×	3/8	,,		21	,,	25
9¼ ×	3⅜ ×	7/16	,,		23½	,,	28
9 ×	5 ×	7/16	,,		34	,,	40
10 ×	5 ×	½	,,		38	,,	48
10 ×	6 ×	½	,,		48	,,	52
10½ ×	4⅜ ×	½	,,		34	,,	40
12 ×	5 ×	½	,,		42	,,	56

PLATES.

Maximum Length..	40′	0″
,, Width	6	8
,, Thickness		1½

FLATS.

3/8 to 5 inches wide; also 5¼, 5⅜, 5½, 5¾, 6, 6¼, 6½, 7, 7¼, 7½, 8, 8¼, 8½, 9, 9¼, 9½, 10, 10¼, 10½, 11, 11½, 12, 12½, 13, and 14, inches wide.

ROUNDS.

	"		"			"
From	¼	to	½	rising by		1/32
,,	½	,,	4	,,	,,	1/16
,,	4	,,	6½	,,	,,	⅛

and 7 inches.

SQUARES.

	"		"			"
From	¼	to	½	rising by		1/32
,,	½	,,	4	,,	,,	1/16
,,	4	,,	6	,,	,,	⅛

and 6½ inches.

Sections of Iron rolled at the Earl of Dudley's Round Oak Iron Works.

FLATS.

2", 2⅛", 2¼", 2⅜", 2½", 2⅝", 2¾", 2⅞", 3", 3⅛", 3¼", 3⅜", 3½", 3¾", 4", 4¼", 4½", 5", 5½", 6", 6¼", 6½", 7", 7½", 7¾", 8", 9", 10", 11" and 12" wide.

ROUNDS.

Up to 6½" diameter.

SQUARES.

Up to 5½" square.

PLATES.

5′ wide from ¼" to 1½" thick.

4′ 2" wide 3/16" and ¼" thick.

TEES.

HEAD.		STEM.	
WIDTH.	THICKNESS.	WIDTH.	THICKNESS.
6"	1/2" & 9/16"	6"	1/2" to 9/16"
6"	3/8", 7/16", 1/2", & 3/4"	3"	3/8" „ 3/4"
5"	1/2" & 9/16"	6"	1/2" „ 9/16"
5"	1/2" & 17/32"	5"	1/2" „ 9/16"
5"	3/8", 7/16", 1/2" & 3/4"	4"	3/8" „ 3/4"
5"	3/8", 1/2" & 5/8"	3 1/2"	3/8" „ 5/8"
5"	3/8", 7/16", 1/2" & 3/4"	3"	3/8" „ 3/4"
5"	5/8" & 1/2"	2 3/4"	3/8" „ 1/2"
5"	3/8", 7/16", 1/2" & 3/4"	2 1/2"	3/8" „ 3/4"
4 1/2"	3/8" & 1/2"	4 1/2"	3/8" „ 1/2"
4 1/2"	3/8" & 1/2"	4"	3/8" „ 1/2"
4 1/8"	3/8"	4"	3/8" „ 1/2"
4"	1/2" & 9/16"	5"	1/2" „ 9/16"
4"	3/8" & 1/2"	4 1/2"	3/8" „ 1/2"
4"	1/2" & 3/4"	4"	1/2" „ 3/4"
4"	3/8" & 1/2"	4"	3/8" „ 1/2"
4"	3/8" & 1/2"	3"	3/8" „ 1/2"
4"	3/8" & 1/2"	2 1/2"	3/8" „ 1/2"
4"	3/8" & 1/2"	2"	3/8" „ 1/2"
3 7/8"	1/2"	3"	3/8"
3 1/2"	3/8", 7/16", 1/2" & 5/8"	3 1/2"	3/8" to 5/8"
3 1/2"	3/8" & 1/2"	3"	3/8" „ 1/2"
3 1/4"	3/8" & 1/2"	3 1/4"	3/8" „ 1/2"
3"	3/8" & 1/2"	5"	3/8" „ 1/2"
3"	3/8" & 1/2" & 5/8"	4"	3/8" „ 5/8"
3"	3/8" & 1/2"	3 1/2"	3/8" „ 1/2"
3"	3/8" & 1/2"	3 1/4"	3/8" „ 1/2"
3"	3/8", 1/2" & 5/8"	3"	3/8" „ 5/8"
3"	1/4" & 5/16"	3"	1/4" „ 5/16"
3"	3/8" & 1/2"	2 1/2"	3/8" „ 1/2"
3"	5/16", 3/8" & 1/2"	2"	5/16" „ 1/2"

ANGLES.

LEGS.	THICKNESSES.
7 " × 3 "	½ " to ¾ "
6 " × 6 "	½ " to 1½ "
6 " × 3 "	⅜ " to ¾ "
5 " × 5 "	$\frac{7}{16}$ " to 1¼ "
5 " × 4 "	⅜ " to ¾ "
5 " × 3½"	⅜ " to ¾ "
5 " × 3¼"	⅜ " to ¾ "
5 " × 3 "	⅜ " to ¾ "
4½" × 4½"	⅜ " to ⅞ "
4½" × 3¾"	⅜ " to ¾ "
4½" × 3 "	⅜ " to ¾ "
4 " × 4 "	⅜ " to ⅞ "
4 " × 3 "	⅜ " to ¾ "
4 " × 2½"	⅜ " to ¾ "
4 " × 2¼"	$\frac{5}{16}$ " to ⅝ "
3½" × 3½"	⅜ " to ¾ "
3½" × 3 "	¼ " to ¾ "
3½" × 2½"	$\frac{5}{16}$ " to ⅝ "
3¼" × 3¼"	⅜ " to ¾ "
3⅛" × 2¾"	$\frac{5}{16}$ " to $\frac{9}{16}$ "
3⅛" × 2⅝"	$\frac{5}{16}$ " to $\frac{9}{16}$ "
3 " × 3 "	¼ " to 1 "
3 " × 2½"	¼ " to ⅝ "
3 " × 2 "	¼ " to ½ "
2¾" × 2¾"	¼ " to ⅝ "
2½" × 2½"	¼ " to ⅝ "
2 " × 2 "	¼ " to ½ "

I BEAMS.

Depth.	Width of Flanges.	Thickness of Web.
12″	6″ to 6 1/4″	1 5/16″ to 1 9/16″
10 3/4″	5 3/4″ „ 6″	1″ „ 1 1/4″
9 1/4″	3 5/8″ „ 3 7/8″	1/2″ „ 3/4″
9″	5″ „ 5 1/4″	7/16″ „ 11/16″
7 1/8″	5 1/8″ „ 5 3/8″	3/4″ „ 1″
5″	4 1/4″ „ 4 1/2″	1/2″ „ 3/4″
4″	3″ „ 3 1/4″	1/2″ „ 3/4″

Sections of Iron rolled by the Société Cockerill at Seraing, Belgium.

ANGLES.

Legs.	Thickness.	Weight per ft.	Legs.	Thickness.	Weight per ft.
5 7/8″ × 5 7/8″	9/16″	22.85″	3 3/8″ × 2 9/16″	5/16″	6.87″
5 7/8″ × 4 1/8″	1/2″	17. 5″	3 1/8″ × 3 1/8″	1/4″	6. 7″
5 7/8″ × 3 1/8″	3/8″	13. 5″	3 1/8″ × 2 3/8″	5/16″	5.74″
5 1/8″ × 3 15/16″	3/4″	22.85″	3″ × 2 1/8″	3/8″	7. 4″
5 1/8″ × 3 1/8″	1/2″	15.15″	2 15/16″ × 2 15/16″	1/4″	6.07″
5 1/8″ × 3 1/8″	3/8″	12. 8″	2 13/16″ × 2 3/4″	5/16″	6. 7″
4 11/16″ × 3 1/8″	3/8″	10. 1″	2 15/16″ × 1 3/4″	1/4″	3.88″
4 3/4″ × 4 3/4″	1/2″	14. 5″	2 3/4″ × 2 3/4″	1/4″	5.07″
4 5/16″ × 4 5/16″	3/8″	11. 1″	2 3/4″ × 2 1/2″	1/4″	5.07″
4″ × 3″	5/16″	8.07″	2 3/4″ × 2 5/16″	1/4″	4.38″
3 11/16″ × 3 1/8″	1/2″	10.77″	2 3/4″ × 1 15/16″	1/4″	3.77″
3 7/8″ × 3 7/8″	3/8″	10. 1″	2 3/4″ × 1 7/16″	7/16″	6.07″
3 7/8″ × 3 9/16″	3/8″	9.43″	2 9/16″ × 2 9/16″	1/4″	4.72″
3 9/16″ × 3 9/16″	5/16″	8.07″	2 1/2″ × 1 3/4″	1/4″	3.37″
3 9/16″ × 3 3/4″	5/16″	7. 4″	2 1/8″ × 2 1/8″	1/4″	3.54″
3 3/8″ × 3 3/8″	3/8″	7. 4″			

N.B.—The minimum thicknesses are given: they can be increased by nearly fifty per cent.

I BEAMS.

Depth.	Width.	Thickness.	Weight per foot.
12 1/8″	6 1/4″	11/16″	60.7
12″	5 13/16″	5/8″	57.1
11 13/16″	4 15/16″	7/16″	39.8
9 7/8″	4 1/8″	7/16″	30.25
9 7/8″	3 5/8″	5/16″	20.15
9 1/2″	3 3/4″	3/8″	23.2
9 1/4″	3 1/8″	3/8″	22.15
8 5/8″	2 3/4″	5/16″	16.83
7 7/8″	4 15/16″	1/2″	26.9
7 7/8″	3 7/8″	1/2″	20.85
7 7/8″	2 3/16″	5/16″	15.47
7 1/8″	3 7/8″	1/2″	20.15
7 1/8″	3 1/8″	5/16″	17.5
7 1/16″	2 5/8″	1/4″	12.4
6 5/16″	3 1/8″	5/16″	15.47
6 5/16″	2 1/8″	1/4″	10.7
5 1/8″	2 3/4″	1/4″	11.43
5 1/8″	2″	1/4″	8.7
5″	2 15/16″	1/4″	11.43
4 15/16″	2″	1/4″	8.0

N. B. The widths and thicknesses are given at their minima, and can be increased by a quarter of an inch or less.

TEES.

Head.	Stem.	Thickness.	Weight per foot.
7 1/4"	4 5/16"	3/8"	24.9 *
6 3/4"	4 1/2"	7/16"	20.8 *
6 3/8"	3 13/16"	7/16"	17.5 *
6 5/16"	4 5/16"	1/2"	23.5 *
6"	4 1/2"	3/8"	18.8 *
5 13/16"	3 13/16"	7/16"	16.5 *
5 13/16"	3 1/2"	7/16"	14.0 *
5 1/2"	3"	1/4"	10.2 *
5 3/16"	2 3/8"	3/8"	12.1 *
5 1/8"	2 13/16"	1/4"	12.1 *
4 15/16"	2 3/8"	3/8"	14.5 *
4 13/16"	2 5/16"	1/4"	8.07*
4 7/16"	3 1/4"	1/2"	17.83*
4 7/16"	3 3/16"	3/8"	15.82*
4 5/16"	2 13/16"	7/16"	10.4 *
3 13/16"	3 1/2"	7/16"	10.77*
3 3/8"	2 3/4"	1/4"	7.74*
3 3/16"	2 1/4"	5/16"	5.73*

N. B. The dimensions of the tees are invariable.

CHANNELS.

Depth.	Width of Flanges.	Thickness of Web.	Weight per foot.
11 13/16"	2 15/16"	1/2"	23.5*
10 3/4"	3 3/16"	1/2"	22.1*
9 7/8"	3 3/16"	7/16"	26.0*
9 7/8"	3 3/16"	1/2"	23.5*
9 1/8"	3 1/8"	5/16"	21.2*
9 3/4"	3 3/16"	1/2"	21.5*
7 7/8"	3 13/16"	7/16"	26.4*
7 7/8"	3 3/16"	7/16"	18.8*
6 7/8"	2 1/8"	5/16"	13.1*
5 11/16"	2 5/16"	5/16"	10.8*
4 5/8"	2 3/16"	1/2"	12.1*
4 1/8"	2 3/16"	5/16"	6.9*

N. B. The widths and thicknesses are given at their minima, and can be increased by a quarter of an inch or less.

Sections of Iron Rolled by De Leeuw and Phillipsen, Antwerp, Belgium.

I BEAMS.

Depth.	Width.	Thicks.	Wt. per ft.	Depth.	Width.	Thicks.	Wt. per ft.
21 5/8″	7 7/8″	3/4″	111.5#+	9 1/4″	3 1/2″ to 3 25/32″	5/16″ to 19/32″	19# to 27.5#
19 3/4″	7″	22/32	91.5#+	9″	4″ to 4 7/32″	3/8″ to 19/32″	23# to 29.5#
17 3/4″	6 5/8″	11/16″	77#+	8 7/8″	6 1/8″ to 6 11/32″	19/32″ to 13/16″	39# to 45#
16 3/4″	6 5/16″	11/16″	70#+	8 3/4″	3 3/4″ to 4″	3/8″ to 5/8″	22.5# to 29.5#
16″	6″ to 6 5/16″	9/16″ to 3/4″	57# to 67#	8″	6″ to 6 1/8″	1/2″ to 5/8″	33# to 37#
15 3/4″	6″ to 6 3/16″	9/16″ to 3/4″	57# to 67#	8″	5″ to 5 1/8″	3/8″ to 3/4″	27.5# to 37.5#
15 3/4″	5 1/2″ to 5 13/16″	9/16″ to 7/8″	57# to 68.5#	7 7/8″	5″ to 5 1/8″	3/8″ to 9/16″	26# to 34.5#
15″	5 1/2″ to 5 5/8″	19/32″ to 23/32″	61# to 67#	8″	4″ to 4 1/16″	3/8″ to 1/2″	28.3# to 33.5#
15″	5″ to 5 3/8″	1/2″ to 7/8″	50.5# to 67#	8″	4″ to 4 7/16″	3/8″ to 13/16″	21.5# to 33.5#
14″	6″ to 6 1/4″	9/16″ to 13/16″	57# to 67#	7 7/8″	4″ to 4 1/4″	3/8″ to 5/8″	20.3# to 27#
14″	6″ to 6 1/4″	9/16″ to 13/16″	53.8# to 64#	7 1/4″	3 1/2″ to 3 11/16″	5/16″ to 5/8″	15.8# to 23#
14″	5 1/2″ to 5 3/4″	9/16″ to 13/16″	53.8# to 65.8#	7 1/8″	4″ to 4 1/4″	3/8″ to 5/8″	20.3# to 27#
13 3/4″	5 1/2″ to 5 21/32″	19/32″ to 13/16″	53.8# to 63#	7″	3 5/8″ to 4″	5/16″ to 7/16″	18.3# to 24.3#
12 1/2″	6 1/4″ to 6 5/16″	11/16″ to 3/4″	60.5# to 63.8#	7″	3 1/2″ to 3 3/4″	5/16″ to 9/16″	17.3# to 23.5#
12 1/2″	5 5/8″ to 5 13/16″	5/8″ to 13/16″	53.5# to 59.8#	6 7/8″	4″ to 4 1/8″	5/16″ to 1/2″	19.5# to 24.3#
12 5/8″	5 3/8″ to 5 9/16″	5/8″ to 7/8″	50.8# to 63#	6 7/8″	3 1/8 to 3 3/8″	5/16″ to 7/16″	14.3# to 17.8#
12 5/8″	5 5/16″ to 5 7/16″	7/16″ to 9/16″	43.5# to 48.5#	6 1/2″	3 1/2″ to 3 3/4″	9/32″ to 1/2″	16# to 21#
12″	7 1/4″ to 7 1/2″	3/4″ to 1″	70# to 80.5#	6 1/4″	3 1/8″ to 3 7/16″	5/16″ to 5/8″	15.5# to 24.3#
12″	6″ to 6 3/8″	1/2″ to 7/8″	51# to 67#	6″	5″ to 5 1/4″	3/8″ to 5/8″	25.3# to 31#
12″	6″ to 6 3/16″	1/2″ to 11/16″	47# to 55#	6″	3″ to 3 1/4″	5/16″ to 9/16″	14.3# to 19#
12″	5″ to 5 3/8″	7/16″ to 13/16″	40.3# to 57#	6″	3 5/8″ to 3 13/16″	7/16″ to 5/8″	22.3# to 25.3#
11″	5″ to 5 3/4″	7/16″ to 13/16″	36.3# to 49.8#	5 15/16″	3 1/8″ to 3 11/32″	9/32″ to 1/2″	13.3# to 17.3#
10″	6″ to 6 1/16″	5/8″ to 11/16″	54.5# to 57#	5 3/4″	3 7/16″ ,, 3 5/8″	7/16″ to 5/8″	21.3# to 24.5#
10″	5″ to 5 1/2″	7/16″ to 13/16″	34# to 47#	5 1/2″	3″ to 3 3/16″	1/4″ to 7/16″	12.3# to 16.3#
10″	4 1/2″ to 4 13/16″	7/16″ to 3/4″	30.3# to 43#	5 1/2″	2 3/4″ to 3	1/4″ to 1/2″	12.3# to 16.3#
10″	4″ to 4 5/16″	13/32″ to 22/32″	28.5# to 39#	5 1/8″	3 1/4″ to 3 1/2″	5/16″ to 9/16″	13.5# to 18.3#
10″	4″ to 4 3/16″	5/16″ to 1/2″	22.5# to 27.5#	5″	4 1/2″ to 4 3/4″	3/8″ to 5/8″	21.5# to 27#
9 1/2″	4 3/4″ to 4 29/32″	7/16″ to 19/32″	34.3# to 39.5#	5″	3″ to 3 7/32″	7/32″ to 7/16″	11.8# to 15.5#
9 1/2″	4 1/2″ to 4 27/32″	3/8″ to 22/32″	27# to 39#	5″	3 5/8″ to 3 7/8″	3/8″ to 21/32″	17.5# to 21#
9 1/2″	3 3/4″ to 4 1/16″	3/8″ to 11/16″	23# to 33.5#	4 3/4″	3 1/8″ to 3 7/16″	5/16″ to 5/8″	12.3# to 16.3#
9 5/16″	3 7/16″ to 3 11/16″	11/32″ to 19/32″	21.3# to 28.3#	4 3/4″	3″ to 3 1/4″	9/32″ to 17/32″	10.8# to 16.5#
9 1/4″	3 3/4″ to 4 1/16″	3/8″ to 11/16″	23# to 33.5#	4 3/4″	2 3/4″ to 3 1/32″	1/4″ to 17/32″	9.8# to 13.5#
9 1/4″	3 3/4″ to 4″	3/8″ to 5/8″	21# to 29.5#	4″	3″ to 3 3/16″	1/4″ to 7/16″	11.5# to 13.5#
9 1/4″	3 9/16″ to 4″	3/8″ to 13/16″	21.5# to 33.5#	4″	2 5/8″ to 2 3/4″	1/4″ to 1/2″	8# to 10#

N. B. Beams can be rolled to any thicknesses and weights between the minimum and maximum sizes.

CHANNELS.

Depth.	Width.	Thickness.	Wt. per ft.	Depth.	Width.	Thickness.	Wt. per ft.
14"	4"	⅝"	41•	5¾"	3½"	½"	18.3•
12"	3 13/16"	½"	31.5•	5¾"	2 13/16"	⅜"	12.8•
12"	3"	⅜"	23.5•	5¾"	2⅜"	5/16"	10.8•
10½"	3½"	⅜"	22.3•	5½"	2¼"	⅜"	12.5•
10"	2 13/16"	⅜"	19.5•	5½"	2"	½"	12•
9⅞"	3 3/16"	7/16"	26.3•	5½"	2"	9/32"	8•
9⅞"	3⅛"	⅜"	22•	5 7/16"	2 11/16"	7/16"	14.8•
8¾"	2¾"	⅜"	18.8•	5¼"	1¾"	⅜"	11.5•
8½"	3 7/16"	9/16"	28.5•	5⅛"	1¾"	9/32"	7.8•
8¾"	3 13/16"	⅜"	23•	5"	2 13/16"	⅝"	18.3•
8"	3 7/16"	⅜"	21.5•	4 15/16"	2¾"	5/16"	12.5•
8"	2"	9/32"	12•	4⅞"	3"	7/16"	15•
7⅞"	3¾"	7/16"	21.3•	4¾"	1¾"	¼"	9.8•
7⅞"	3½"	⅜"	18.8•	4⅝"	2 13/16"	⅜"	12.3•
7¾"	3 1/16"	½"	27.3•	4⅝"	2⅛"	⅜"	11.5•
7¾"	2⅝"	5/16"	12.8•	4 9/16"	2¾"	7/16"	14•
7"	2⅞"	⅜"	16.3•	4½"	2"	9/32"	7•
6⅞"	2⅛"	5/16"	12.8•	4½"	2⅞"	⅜"	12.5•
6 15/16"	2 13/16"	5/16"	12.5•	4 3/16"	2"	⅜"	10.8•
6 15/16"	2⅛"	5/16"	12.3•	4½"	2 13/16"	5/16"	10•
6 15/16"	2⅛"	⅜"	14.3•	4½"	2⅛"	9/32"	9.3•
6"	3"	⅜"	14.3•	4½"	2 3/16"	¼"	7.5•
6"	2½"	½"	15•	4"	2½"	5/16"	10.5•
6"	2¼"	9/32"	10.5•	4"	2"	⅜"	5.5•
6"	1⅞"	⅜"	11•	4"	1⅞"	3/16"	9.5•
5⅞"	2⅝"	7/16"	15.8•	4"	1 13/16"	5/16"	7.5•

N.B.—Owing to a mistake of the binder's eight channel sections of depths from 9⅞" to 8¾" were omitted from the catalogue from which this table was prepared. The thicknesses of the webs and the weights can be increased.

ANGLES·

Legs.	Thickness.	Weight per foot.
10" × 3½"	5/8"	26.5•
8½" × 4 1/16"	15/32"	20•
8" × 5½"	11/16"	28.5•
8" × 4"	5/8"	23.3•
7½" × 3 15/16"	5/8"	20.3•
7⅛" × 3⅛"	9/16"	18.3•
6 11/16" × 3 5/16"	9/16"	19•
6 5/16" × 5½"	9/16"	21•
6 5/16" × 4 5/16"	9/16"	19•
6½" × 6½"	½"	19.5•
6" × 6"	9/16"	20.8•
6" × 4⅛"	17/32"	18.3•
6" × 4"	9/16"	17.3•
6" × 3½"	15/32"	14.3•
6" × 3⅛"	½"	15.3•
5½" × 5½"	17/32"	18.8•
5½" × 4"	23/32"	22.8•
5½" × 4"	15/32"	14.3•
5½" × 3⅛"	7/16"	12•
5⅛" × 5⅛"	½"	16.8•
5⅛" × 4"	5/8"	18.3•
5⅛" × 3½"	13/32"	11•
5⅛" × 3 1/16"	½"	12.5•
5⅛" × 3"	11/32"	9•
5" × 5"	7/16"	14•
4 15/16" × 4 15/16"	½"	15.5•
4 15/16" × 3½"	15/32"	12.8•
4 15/16" × 3⅛"	11/32"	9.5•
4¾" × 4¾"	15/32"	14.5•
4¾" × 4"	17/32"	14.5•
4¾" × 3½"	17/32"	12.8•
4¾" × 3⅛"	5/16"	8•
4½" × 4½"	7/16"	12.8•
4½" × 3"	11/32"	9.3•
4⅜" × 4⅜"	13/32"	11•
4⅜" × 3½"	⅜"	105•
4⅜" × 2¾"	5/16"	7•
4⅜" × 2½"	13/32"	8.8•
4⅜" × 2"	9/32"	5.8•
4⅛" × 4⅛"	13/32"	10.5•
4⅛" × 3½"	13/32"	10.3•
4⅛" × 3½"	15/32"	11.8•
4⅛" × 3"	13/32"	9•
4⅛" × 2½"	5/16"	6.9•
4⅛" × 2⅛"	5/16"	6.4•
4" × 4"	13/32"	10•
4" × 3½"	13/32"	9.5•
4" × 3⅛"	5/16"	8•
4" × 3⅛"	½" & 7/16"	10.8•
4" × 3"	11/32"	8•
4" × 2¾"	5/16"	6.9•
4" × 2⅛"	11/32"	8•
3¾" × 2½"	9/32"	6•
3¾" × 2⅜"	5/16"	6•
3½" × 3½"	23/32"	17.3•
3½" × 3½"	11/32"	8.3•
3½" × 3"	11/32"	7.5•
3½" × 2¾"	11/32"	7.5•
3½" × 2⅜"	5/16"	6•
3½" × 2⅛"	9/32"	5.1•
3½" × 2"	9/32"	4.5•
3⅜" × 3⅜"	11/32"	7.5•
3⅜" × 2¾"	11/32"	7•
3⅜" × 2½"	11/32"	6.9•
3⅛" × 3⅛"	5/16"	6.4•
3½" × 2⅜"	5/16"	5.4•
3½" × 2"	7/16" & ¼"	6•
3 1/16" × 3 1/16"	5/16"	6.3•
3" × 3"	5/16"	6•
3" × 2¾"	11/32"	6.7•
3" × 2½"	13/32"	7•
3" × 2"	5/16"	5.2•
2¾" × 2¾"	9/32"	4.9•
2¾" × 2 9/16"	9/32"	5•
2¾" × 2⅜"	¼"	3.9•
2¾" × 2"	¼"	3.5•
2 9/16" × 2 9/16"	9/32"	4.6•
2½" × 2½"	⅜"	6.4•
2½" × 2 5/16"	13/32" & 5/16"	5.5•
2½" × 2"	3/16"	3•
2⅜" × 2⅜"	¼"	3.7•
2¼" × 2¼"	¼"	3.4•
2 3/16" × 2 3/16"	3/16"	2.7•
2" × 2"	3/16"	2.5•

TEES.

Head.	Stem.	Wt. per ft.	Head.	Stem.	Wt. per ft.
8 1/2" × [illegible]"	4 3/4" × [illegible]"	29.5	5" × 1 1/8"	3 15/16" × 1 1/8"	35
8 3/16" × [illegible]"	4 7/16" × [illegible]"	25	5" × 9/16"	3 1/2" × 5/16"	15.5
8 1/8" × 13/16"	3 7/8" × [illegible]"	32.3	5" × [illegible]"	2 9/16" × 7/16"	10.8
8 1/16" × [illegible]"	3 11/16" × [illegible]"	28	5" × 5/16"	2 1/2" × 1/2"	8
7 7/8" × 1/2"	4 1/16" × 1/2"	21	4 7/8" × [illegible]"	2 1/2" × [illegible]"	15.3
7 1/8" × 3/4"	4 1/4" × [illegible]"	25	4 3/4" × [illegible]"	3 3/4" × 7/16"	13.8
6 11/16" × [illegible]"	4 7/16" × 1/2"	21	4 3/4" × [illegible]"	4 1/2" × 5/8"	18
6 11/16" × 11/16"	4 5/16" × 5/8"	23	4 3/4" × [illegible]"	3 5/16" × [illegible]"	21
6 11/16" × [illegible]"	3 15/16" × [illegible]"	26.3	4 3/4" × 7/16"	3 1/4" × [illegible]"	10.8
6 11/16" × 1/2"	3 15/16" × [illegible]"	19.5	4 1/2" × 1/2"	5" × [illegible]"	15.5
6 11/16" × 1/2"	3 15/16" × 1/2"	17.5	4 7/16" × [illegible]"	3 1/2" × [illegible]"	16.8
6 7/16" × 13/16"	3 5/8" × 1"	25.5	4 7/16" × 5/16"	3 1/2" × [illegible]"	15.8
6 5/16" × 3/4"	4 1/4" × [illegible]"	23.5	4 5/8" × 1/2"	3 1/4" × 1/2"	12.3
6 5/16" × [illegible]"	3 11/16" × 11/16"	19.5	4 3/8" × [illegible]"	3 1/8" × 1/2"	10.8
6 5/16" × 7/16"	3 5/16" × 7/16"	15.3	4 3/8" × 15/32"	2 5/8" × 15/32"	10.5
6 1/16" × 11/16"	3 5/16" × 13/16"	22	4 3/16" × 13/32"	3 1/16" × 13/32"	9.5
6" × 11/16"	4 1/4" × [illegible]"	28.5	4 1/8" × 9/16"	3 1/16" × 5/16"	13.5
6" × [illegible]"	3 5/16" × 11/16"	18.8	4 1/8 × [illegible]"	3 1/8" × 1/2"	10.5
6" × 1/2"	4 1/2" × 1/2"	18.8	4" × 13/32"	4" × [illegible]"	10
6" × 1/2"	4" × 1/2"	16.3	4" × 1/2"	5" × 1/2"	11
6" × [illegible]"	3 15/16" × 5/8"	15.8	4" × 3/8"	3" × 3/8"	8
6" × 1/2"	3 3/4" × 1/2"	14.8	3 15/16" × 15/32"	6 5/16" × 7/16"	14.5
6" × 11/16"	9" × 5/8"	31.5	3 15/16" × 7/16"	5 11/16" × [illegible]"	12.5
6" × 9/16"	3 9/16" × [illegible]"	16.8	3 15/16" × 7/16"	4 5/16" × 1/2"	12
6" × 1/2"	3 5/16" × 1/2"	14.5	3 15/16" × 5/16"	3 1/4" × [illegible]"	11.5
6" × 1/2"	3 1/2" × 5/16"	15.5	3 15/16" × [illegible]"	3 1/8" × [illegible]"	10.8
6" × 7/16	3 1/2" × 1/2"	13.3	3 15/16" × 3/8"	2 3/4" × 1/2"	8.3
6" × 1/2"	3" × 1/2"	14	3 15/16" × 5/16"	2 1/2" × 5/16"	6.8
6" × 3/8"	3" × 1/2"	11.3	3 15/16" × [illegible]"	2" × 5/8"	10.5
6" × 7/16"	2 5/16" × 1/2"	12	3 7/8" × 9/16"	3 3/16" × 5/16"	12.8
5 15/16" × [illegible]"	4 1/16" × 7/8"	27.8	3 7/8" × 15/32"	3" × [illegible]"	11.5
5 3/4" × 7/8"	3 5/8" × 1"	25	3 7/8" × 7/16"	2 7/16" × 7/16"	9
5 3/4" × [illegible]"	3 5/16" × [illegible]"	19.3	3 3/4" × 7/16"	4 1/2" × 1/2"	12.8
5 11/16" × [illegible]"	4 1/2" × 9/16"	19.5	3 11/16" × 3/8"	2 15/16" × 13/16"	11.5
5 11/16" × [illegible]"	3 7/8" × [illegible]"	21.8	3 11/16" × 5/16"	2 3/4" × 5/16"	6.8
5 11/16" × 1/2"	3" × [illegible]"	16.3	3 5/16" × 7/16"	5 1/8" × [illegible]"	10.3
5 1/2" × 13/16"	3 5/8" × 11/16"	21.3	3 5/8" × [illegible]"	3 1/2" × [illegible]"	8.3
5 1/2" × 7/8"	4 1/2" × 11/16"	24.5	3 1/2" × 7/16"	3 5/16" × [illegible]"	9.3
5 1/2" × 5/16"	3 9/16" × 3/8"	17.3	3 1/2" × [illegible]"	2 3/4" × 3/8"	7.8
5 1/2" × 3/8"	3" × 1/2"	10.8	3 1/2" × 1/4"	2 5/16" × [illegible]"	5.5
5 5/16" × 1/2"	3 1/2" × 5/8"	17.3	3 1/2" × 3/8"	2 7/16" × 3/8"	7.3
5 7/16" × [illegible]"	2 5/8" × 9/16"	12.3	3 1/2" × [illegible]"	1 3/4" × 7/16"	7
5 3/8" × [illegible]"	6 1/4" × [illegible]"	26	3 3/8" × 9/32"	2 7/8" × 9/32"	6.3
5 3/8" × 1/2"	3 5/8" × 1"	20.5	3 1/4" × [illegible]"	2 3/16" × [illegible]"	9.8
5 3/8" × 5/16"	3" × 11/16"	15.8	3 3/8" × 5/16"	1 15/16" × [illegible]"	5.3
5 5/16" × [illegible]"	6" × 1/2"	19.5	3 1/4" × [illegible]"	3 1/4" × [illegible]"	12.5
5 5/16" × [illegible]"	4 1/4" × 5/8"	25.3	3 1/4" × 9/32"	2 1/16" × 9/32"	5
5 1/4" × [illegible]"	4 3/4" × 1 1/16"	28	3 1/4" × 1/2"	1 5/8" × 3/8"	5.8
5 1/4" × [illegible]"	2 3/4" × 1/2"	11.8	3 1/2" × 1/2"	2 5/16" × 1/2"	9.5
5 1/4" × 1/2"	2 5/16" × 9/16"	12	3 1/2" × 7/16"	2 5/16" × 7/16"	8
5 1/8" × [illegible]"	3 5/8" × [illegible]"	17.8	3 1/2" × 5/16"	2 1/2" × 5/16"	5.5
5 1/8" × [illegible]"	6" × 7/8"	26.3	3" × 1/2"	3" × 1/2"	9.5
5 1/8" × 3/8"	3 1/2" × [illegible]"	10.8	3" × 7/16"	3" × 7/16"	8.3
5" × 1/2"	4" × 1/2"	14.3	3" × 3/8"	3" × 3/8"	7
5" × 1/2"	5 1/2" × 1/2"	13.8	3" × 5/16"	2 5/16" × 5/16"	5
5 1/8" × 1/2"	3" × 1/2"	12.3	3" × 15/32"	2 1/4" × 13/32"	7.5
5 1/8" × 3/8"	3" × 3/8"	11	3" × 7/16"	2" × 7/16"	6.9
5 1/2" × [illegible]"	2 5/16" × 7/16"	9.8	3" × 5/16"	2" × 5/16"	5
5" × [illegible]"	6" × [illegible]"	21.3			

Sections of Iron Rolled by the New Jersey Steel and Iron Co. Trenton, New Jersey. U. S. A.

Height in Inches.	Weight per yd. in Pounds.	Width of Flange. Ins.	Thickness of Web, in Ins.
BEAMS.			
15⅛	200	5¾	.6
15 7/16	150	5	½
12 9/16	170	5½	.6
12¼	125	4.79	.47
10½	135	5	.47
10½	105	4½	⅜
10½	90	4½	5/16
9	125	4½	.57
9	85	4	⅜
9	70	3½	.3
8	80	4½	⅜
8	65	4	.3
7	55	3¾	.3
6	120	5¼	⅝
6	90	5	½
6	50	3½	.3
6	40	3	¼
5	40	3	5/16
5	30	2¾	¼
4	37	3	5/16
4	30	2¾	¼
4	18	2	3/16
CHANNELS.			
15	190	4¾	¾
15	120	4	½
12¼	140	4	11/16
12¼	85	3	7/16
10½	60	2¾	⅜
9	70	3⅛	7/16
9	50	2½	.33
8	45	2½	.26
8	33	2.2	.20
7	36	2½	¼
7	25½	2.0	.20
6	45	2½	.40
6	33	2¼	.28
6	22½	1⅞	.18
5	19	1⅝	.20
4	16½	1½	.20
3	15	1½	.20

ANGLES.

Designation of Bar.	Approximate Weight, in pounds, per yard, for each thickness in inches.							
	7/16	½	9/16	⅝	11/16	¾	13/16	⅞
6 x 6........	50.6	57.5	64.3	71.1	77.8	84.4	91.0	97.3
4½ x 4½.....	37.5	42.5	47.5	52.3	57.2	61.9	......	......
	⅜	7/16	½	9/16	⅝	11/16	¾	
4 x 4........	28.6	33.1	37.5	41.8	46.1	50.5	54.4	......
3½ x 3½.....	24.8	28.7	32.5	36.2	39.8	43.4	......	......
	¼	5/16	⅜	7/16	½	9/16	⅝	11/16
3 x 3........	14.4	17.7	21.1	24.4	27.5	30.6	33.6	36.5
	5/16	⅜	13/32	7/16	15/32	½	17/32	9/16
2¾ x 2¾.....	16.2	19.2	20.7	22.2	23.6	25.0	26.3	27.7
	¼	5/16	11/32	⅜	13/32	7/16	15/32	½
2½ x 2½.....	11.9	14.7	16.0	17.3	18.6	20.0	21.2	22.5
	¼	9/32	5/16	11/32	⅜	13/32	7/16	
2¼ x 2¼.....	10.6	11.9	13.1	14.3	15.5	16.8	17.8	......
2 x 2........	9.4	10.4	11.5	12.6	13.6	......	......	......
	3/16	7/32	¼	9/32	5/16			
1¾ x 1¾.....	6.21	7.18	8.13	9.05	9.96	......	......	......
1½ x 1½.....	5.27	6.09	6.88	7.64	8.40	......	......	......
	⅛	5/32	3/16	7/32	¼			
1¼ x 1¼.....	2.97	3.66	4.34	4.99	5.63	......	......	......
1 x 1........	2.34	2.88	3.40	3.91	4.38	......	......	......
⅞ x ⅞........	2.03	2.48	2.93	......	......	......	......	......
¾ x ¾........	1.72	2.09	2.46	......	......	......	......	......

UNEVEN LEGS.

	⅜	7/16	½	9/16	⅝	11/16	¾
6 x 4........	......	41.8	47.5	53.1	58.6	64.0	69.4
5 x 3½......	30.5	35.3	40.0	44.7	49.2	53.7	58.1
4½ x 3......	26.7	30.9	35.0	39.0	43.0	......	.....
	5/16	⅜	7/16	½	9/16	⅝	11/16
4 x 3........	20.9	24.8	28.7	32.5	36.2	39.8	43.4
	5/16	11/32	⅜	13/32	7/16	½	9/16
3 x 2½......	16.2	17.7	19.2	20.7	22.2	25.0	27.7
	7/32	¼	9/32	5/16	⅜	7/16	½
3 x 2.......	10.4	11.9	12.3	14.6	17.3	20.0	22.5

TEES.

Designation of Bar.	Approximate Weight, in pounds, per yard, for each thickness in inches.	
4″ x 4″.......	7/16″........33.1 lbs.	½ ″........37.5 lbs.
3½″ x 3½″....	7/16″........28.7 "	½ ″........32.5 "
3″ x 3″.......	....	½ ″........27.5 "
2½″ x 2½″....	5/16″........14.7 lbs.	⅜ ″........17.3 "
2″ x 2″.......	¼ ″........ 9.4 "	5/16″........11.5 "
5″ x 2½″......	7/16″........30.9 "	½ ″........35.0 "
3″ x 2″.......	5/16″........14.6 "	⅜ ″........17.3 "
2¼″ x 1⅛″....	¼ ″........ 7.4 "	....
2″ x 1″.......	¼ ″........ 6.5 "	....

Flats, all Sizes. Rounds, up to 5″ dia.

N. B. It will be noticed that the weights per yard of the Trenton sections are given instead of the weights per foot, this being the method employed by the company.

Sections of Iron Rolled by the Passaic Rolling Mill Co. Patterson, New Jersey. U. S. A.

I BEAMS.

Height in Inches.	Weight per Yd. in Pounds.	Width of Flange in Inches.	Thickness of Web in Inches.
15⅛	200	5½	.6
15 3-16	150	5	½
12 5-16	170	5½	.6
10¾	125	4.79	.48
10½	135	5	.47
10½	105	4½	⅜
10½	90	4⅜	.35
9	85	4	.38
9	70	3½	.3
8	80	4½	.37
8	65	4	.3
7	60	3½	⅜
6	50	3½	.3
6	40	3	¼
5	40	3	5-16
5	30	2¾	¼
4	37	3	5-16
4	30	2¾	¼
4	18	2¼	5-32

ANGLES.

Equal Sides.

Size—Inches.	Thickness—Inches.	Wt. per Ft., in lbs.
¾ × ¾	⅛ to 3-16	¾ to 1
⅞ × ⅞	⅛ to 3-16	⅞ to 1⅛
1 × 1	⅛ to ¼	⅞ to 1¼
1¼ × 1¼	3-16 to 5-16	1¼ to 1¾
1½ × 1½	3-16 to 5-16	1¾ to 2½
1¾ × 1¾	3-16 to ⅜	2 to 4
2 × 2	¼ to ½	3⅛ to 6
2¼ × 2¼	¼ to ½	4 to 7
2½ × 2½	¼ to ⅝	4½ to 10
3 × 3	¼ to ¾	6 to 12
3½ × 3½	5-16 to ¾	7½ to 15
4 × 4	⅜ to ¾	10 to 18
5 × 5	½ to ¾	15 to 20
6 × 6	½ to ¾	19 to 30

Unequal Sides.

Size—Inches.	Thickness—Inches.	Wt. per Ft., in lbs.
1⅜ × 1⅛	3-16 to 5-16	1¼ to 1¾
2 × 1⅜	3-16 to 5-16	2 to 4
2¼ × 1½	¼ to ⅜	3 to 6
2¾ × 2	¼ to ⅜	4½ to 7
4 × 3	⅜ to ¾	8 to 16
4 × 3½	⅜ to ¾	9 to 18
4½ × 3	⅜ to ¾	9 to 18
5 × 3	⅜ to ¾	10 to 18
5 × 3½	⅜ to ¾	11 to 20

TEE IRON.

Equal.

Size—Inches.	Thickness—Inches.	Wt. per Ft., in lbs.
1 × 1	⅛	1
1¼ × 1¼	3-16	1½
1½ × 1½	3-16 & ¼	1¾ to 2
1¾ × 1¾	3-16 & ¼	2 to 3
2 × 2	¼ & 5-16	3 to 5
2½ × 2½	5-16 & ⅜	5 to 6
3 × 3	–3. 7-16 & ½	7 to 10
3½ × 3½	7-16 & ½	9 to 11
4 × 4	7-16 & ½	11 to 13

Unequal.

Size—Inches.	Thickness—Inches.	Wt. per Ft., in lbs.
2¼ × 1¼	¼	3
3 × 2	⅜	6
4 × 2	⅜	7½
5 × 2½	⅜ to ½	9 to 13
5 × 3	⅜ to ½	10 to 12
6 × 4	½ to ⅝	16 to 20
3 × 4	⅜	11

CHANNEL BARS.

Depth Inches.	Flange Inches.	Wt. per yd. in lbs.
6 –	2 to 2⅝	22½ to 56
8 –	2 to 2⅜	30 to 50
9 –	2½ to 3⅛	45 to 70
12¼ –	3 to 4	85 to 140
15 3-16 –	4 to 4¼	150 to 200

Rounds and Square......... ¼″ to 4¾″

Flats............... ¾″ to 8″ wide

N. B. The author has heard that the Passaic Rolling Mill Co. has increased the number of its shapes, but he has not yet seen a copy of its new album. The weights per yard are given here also.

Sections of Iron Rolled by Carnegie Bros. & Co.,
Union Iron Mills Pittsburgh. Pa. U. S. A.

I BEAMS.

Designation.	Weight per foot. Lbs.	Area of Section. Sq. In.	Thickness of Web. In.	Width of Flange. In.	Increase of thickness of web for each lb. increase of weight. In.
15" Light,	50.	15.0	.47	5.03	.02
15" Heavy,	65.	19.5	.77	5.33	
15" Light,	67.	20.1	.67	5.55	.02
15" Heavy,	80.	24.0	.93	5.81	
12" Light,	42.	12.6	.51	4.64	.025
12" Heavy,	60.	18.0	.96	5.09	
10½" Light,	31.5	9.5	.41	4.54	.029
10½" Heavy,	45.	13.5	.79	4.92	
10" Light,	30.	9.0	.32	4.32	.03
10" Heavy,	45.	13.5	.77	4.77	
9" Light,	23.5	7.0	.26	4.01	.033
9" Heavy,	33.	9.9	.58	4.33	
9" Light, Extra,	45.	13.5	.75	4.94	.033
9" Heavy „	50.	15.0	.91	5.10	
8" Light,	22.	6.6	.31	3.81	.038
8" Heavy,	35.	10.5	.79	4.29	
7" Light,	18.	5.4	.23	3.61	.043
7" Heavy,	25.	7.5	.53	3.91	
6" Light,	13.5	4.1	.24	3.24	.05
6" Heavy,	18.	5.4	.46	3.46	
5" Light,	10.	3.0	.225	2.73	.06
5" Heavy,	13.	3.9	.405	2.91	
4" Light,	8.	2.4	.23	2.48	.075
4" Heavy,	10.	3.0	.38	2.63	

CHANNEL BARS.

Designation.	Weight per foot. Lbs.	Area of Section. Sq. In.	Thickness of Web. In.	Width of Flange. In.	Increase of thickness of web for each lb. increase of weight. In.
15" Light,	40.	12.00	.525	3.53	.0200
15" Heavy,	60.	18.00	.925	3.93	
12" One Weight	20.	6.00	.318	3.01	
12" Light,	22.5	6.75	.324	3.01	.0250
12" Heavy,	30.	9.00	.512	3.20	
12" Light,	30.	9.00	.457	2.71	.0250
12" Heavy,	50.	15.00	.957	3.21	
10" One Weight	16.	4.80	.329	2.52	
10" Light,	17.5	5.25	.300	2.45	.0300
10" Heavy,	30.	9.00	.675	2.80	
10" Light,	20.	6.00	.305	2.56	.0300
10" Heavy,	35.	10.50	.755	3.01	
9" One Weight	14.5	4.35	.316	2.50	
9" Light,	18.	5.40	.305	2.43	.0333
9" Heavy,	30.	9.00	.705	2.83	
8" Light,	12.5	3.75	.264	2.01	.0375
8" Heavy,	15.5	4.65	.376	2.13	
8" Light,	16.	4.80	.303	2.30	.0375
8" Heavy,	28.	8.40	.753	2.75	
7" Light,	10.5	3.15	.247	2.00	.0429
7" Heavy,	13.5	4.05	.375	2.13	
7" Light,	14.	4.20	.296	2.30	.0429
7" Heavy,	20.	6.00	.554	2.55	
6" Light,	7.5	2.25	.196	1.76	.0500
6" Heavy,	9.5	2.85	.296	1.86	
6" Light,	10.	3.00	.227	1.98	.0500
6" Heavy,	16.	4.80	.527	2.28	
5" Light,	6.5	1.95	.219	1.66	.0600
5" Heavy,	8.5	2.55	.339	1.78	
5" Light,	9.	2.70	.245	1.93	.0600
5" Heavy,	14.	4.20	.545	2.23	
4" Light,	6.	1.80	.246	1.62	.0750
4" Heavy,	7.	2.10	.321	1.70	
4" Light,	7.	2.10	.244	1.74	.0750
4" Heavy,	9.	2.70	.394	1.89	

ANGLE IRONS.

Weights per Foot corresponding to thicknesses varying by $\frac{1}{16}$". One cubic foot weighing 480 lbs.

Size. Inches.	1/8"	3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	9/16"	5/8"	11/16"	3/4"	13/16"	7/8"
Equal Legs.													
6 × 6	..	..	..	..	..	...	19.2	21.7	24.2	26.7	29.2	31.7	34.2
4 × 4	..	..	..	..	9.5	11.2	12.9	14.5	16.2	17.9	19.5	..	..
3½ × 3½	..	..	..	..	8.3	9.7	11.2	12.7	14.1	15.6	17.0	..	..
3¼ × 3¼	..	..	..	..	7.7	9.0	10.4	11.7	13.1	14.4	15.8	..	..
3 × 3	..	..	..	5.9	7.2	8.4	9.7	10.9	12.2	..	..	..	..
2¾ × 2¾	..	..	..	5.4	6.5	7.7	8.8	..	..	..	..	..	..
2½ × 2½	..	..	..	4.9	5.9	7.0	8.0	..	..	..	..	..	..
2¼ × 2¼	..	..	3.5	4.5	5.4	6.4	7.3	..	..	..	..	..	..
2 × 2	..	..	3.1	4.0	4.8	5.6	..	..	..	..	..	..	..
1¾ × 1¾	..	2.1	2.8	3.5	4.3	5.0	..	..	..	..	..	..	..
1½ × 1½	..	1.8	2.4	3.0	3.6	..	..	..	..	..	..	..	..
1¼ × 1¼	1.0	1.5	2.0	..	..	..	..	..	..	..	..	..	..
1⅛ × 1⅛	0.9	1.4	1.8	..	..	..	..	..	..	..	..	..	..
1 × 1	0.8	1.2	1.6	..	..	..	..	..	..	..	..	..	..
¾ × ¾	0.6	0.9	..	..	..	..	..	..	..	..	..	..	..
Unequal Legs.													
6 × 4	..	..	..	..	..	13.9	16.0	18.1	20.2	22.3	24.4	26.4	..
5 × 4	..	..	..	..	10.8	12.7	14.5	16.4	18.3	20.2	22.0	..	..
5 × 3½	..	..	..	..	10.2	11.9	13.7	15.5	17.2	19.0	20.8	..	..
5 × 3	..	..	..	..	9.5	11.2	12.9	14.5	16.2	17.9	19.5	..	..
4 × 3½	..	..	..	..	8.9	10.5	12.0	13.6	15.2	16.7	18.3	..	..
4 × 3	..	..	..	..	8.3	9.7	11.2	12.7	14.1	15.6	17.0	..	..
3½ × 3	..	..	..	..	7.7	9.0	10.4	11.7	13.1	14.4	15.8	..	..
3¼ × 2	..	..	4.2	5.3	6.4	7.4	8.5	..	..	..	..	..	..
3 × 2½	..	..	4.4	5.5	6.7	7.8	9.0	..	..	..	..	..	..
3 × 2	..	..	4.0	5.0	6.0	7.1	8.1	..	..	..	..	..	..
2½ × 2	..	..	3.5	4.5	5.4	6.4	7.3	..	..	..	..	..	..
2 × 1½	..	..	2.6	3.3	4.0	..	..	..	..	..	..	..	..

T IRONS.

Size, Flange by Stem. Inches.	Weight per Ft. Lbs.	Area of Section Square Inches.
5 × 3	13	3.90
5 × 2½	10¼	3.08
4½ × 3½	15	4.50
4 × 5	14	4.20
4 × 4½	13½	4.05
4 × 4	12	3.60
4 × 3	9¼	2.78
4 × 2½	7½	2.25
4 × 2	6½	1.95
3½ × 4	11¼	3.38
3½ × 3½	10	3.00
3½ × 3	9¼	2.78
3 × 4	12¼	3.68
3 × 3½	11¾	3.53
3 × 3	7.6	2.28
3 × 2½	6	1.80
2½ × 3	6½	1.95
2½ × 2¾	6.6	1.98
2½ × 2½	5.4	1.67
2½ × 1¼	3	0.90

WEIGHTS OF FLAT ROLLED IRON PER LINEAL FOOT.

For Thicknesses from $\frac{1}{16}$ in. to 2 in., and widths from 1 in. to 12¾ in.

Iron weighing 480 lbs. per cubic foot.

Thickness in Inches.	1"	1¼"	1½"	1¾"	2"	2¼"	2½"	2¾"	3"	3¼"	3½"	3¾"	4"	4¼"	4½"	4¾"
$\frac{1}{16}$	.208	.260	.313	.365	.417	.469	.521	.573	.625	.677	.729	.781	.833	.885	.938	.990
$\frac{1}{8}$	.417	.521	.625	.729	.833	.938	1.04	1.15	1.25	1.35	1.46	1.56	1.67	1.77	1.88	1.98
$\frac{3}{16}$	.625	.781	.938	1.09	1.25	1.41	1.56	1.72	1.88	2.03	2.19	2.34	2.50	2.66	2.81	2.97
$\frac{1}{4}$	.833	1.04	1.25	1.46	1.67	1.88	2.08	2.29	2.50	2.71	2.92	3.13	3.33	3.54	3.75	3.96
$\frac{5}{16}$	1.04	1.30	1.56	1.82	2.08	2.34	2.60	2.86	3.13	3.39	3.65	3.91	4.17	4.43	4.69	4.95
$\frac{3}{8}$	1.25	1.56	1.88	2.19	2.50	2.81	3.13	3.44	3.75	4.06	4.38	4.69	5.00	5.31	5.63	5.94
$\frac{7}{16}$	1.46	1.82	2.19	2.55	2.92	3.28	3.65	4.01	4.38	4.74	5.10	5.47	5.83	6.20	6.56	6.93
$\frac{1}{2}$	1.67	2.08	2.50	2.92	3.33	3.75	4.17	4.58	5.00	5.42	5.83	6.25	6.67	7.08	7.50	7.92
$\frac{9}{16}$	1.88	2.34	2.81	3.28	3.75	4.22	4.69	5.16	5.63	6.09	6.56	7.03	7.50	7.97	8.44	8.91
$\frac{5}{8}$	2.08	2.60	3.13	3.65	4.17	4.69	5.21	5.73	6.25	6.77	7.29	7.81	8.33	8.85	9.38	9.90
$\frac{11}{16}$	2.29	2.86	3.44	4.01	4.58	5.16	5.73	6.30	6.88	7.45	8.02	8.59	9.17	9.74	10.31	10.89
$\frac{3}{4}$	2.50	3.13	3.75	4.38	5.00	5.63	6.25	6.88	7.50	8.13	8.75	9.38	10.00	10.63	11.25	11.88
$\frac{13}{16}$	2.71	3.39	4.06	4.74	5.42	6.09	6.77	7.45	8.13	8.80	9.48	10.16	10.83	11.51	12.19	12.86
$\frac{7}{8}$	2.92	3.65	4.38	5.10	5.83	6.56	7.29	8.02	8.75	9.48	10.21	10.94	11.67	12.40	13.13	13.85
$\frac{15}{16}$	3.13	3.91	4.69	5.47	6.25	7.03	7.81	8.59	9.38	10.16	10.94	11.72	12.50	13.28	14.06	14.84
1	3.33	4.17	5.00	5.83	6.67	7.50	8.33	9.17	10.00	10.83	11.67	12.50	13.33	14.17	15.00	15.83
1 $\frac{1}{16}$	3.54	4.43	5.31	6.20	7.08	7.97	8.85	9.74	10.63	11.51	12.40	13.28	14.17	15.05	15.94	16.82
1 $\frac{1}{8}$	3.75	4.69	5.63	6.56	7.50	8.44	9.38	10.31	11.25	12.19	13.13	14.06	15.00	15.94	16.88	17.81
1 $\frac{3}{16}$	3.96	4.95	5.94	6.93	7.92	8.91	9.90	10.89	11.88	12.86	13.85	14.84	15.83	16.82	17.81	18.80
1 $\frac{1}{4}$	4.17	5.21	6.25	7.29	8.33	9.38	10.42	11.46	12.50	13.54	14.58	15.63	16.67	17.71	18.75	19.79
1 $\frac{5}{16}$	4.37	5.47	6.56	7.66	8.75	9.84	10.94	12.03	13.13	14.22	15.31	16.41	17.50	18.59	19.69	20.78
1 $\frac{3}{8}$	4.58	5.73	6.88	8.02	9.17	10.31	11.46	12.60	13.75	14.90	16.04	17.19	18.33	19.48	20.63	21.77
1 $\frac{7}{16}$	4.79	5.99	7.19	8.39	9.58	10.78	11.98	13.18	14.38	15.57	16.77	17.97	19.17	20.36	21.56	22.76
1 $\frac{1}{2}$	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	15.00	16.25	17.50	18.75	20.00	21.25	22.50	23.75
1 $\frac{9}{16}$	5.21	6.51	7.81	9.11	10.42	11.72	13.02	14.32	15.63	16.93	18.23	19.53	20.83	22.14	23.44	24.74
1 $\frac{5}{8}$	5.42	6.77	8.13	9.48	10.83	12.19	13.54	14.90	16.25	17.60	18.96	20.31	21.67	23.02	24.38	25.73
1 $\frac{11}{16}$	5.63	7.03	8.44	9.84	11.25	12.66	14.06	15.47	16.88	18.28	19.69	21.09	22.50	23.91	25.31	26.72
1 $\frac{3}{4}$	5.83	7.29	8.75	10.21	11.67	13.13	14.58	16.04	17.50	18.96	20.42	21.88	23.33	24.79	26.25	27.71
1 $\frac{13}{16}$	6.04	7.55	9.06	10.57	12.08	13.59	15.10	16.61	18.13	19.64	21.15	22.66	24.17	25.68	27.19	28.70
1 $\frac{7}{8}$	6.25	7.81	9.38	10.94	12.50	14.06	15.63	17.19	18.75	20.31	21.88	23.44	25.00	26.56	28.13	29.69
1 $\frac{15}{16}$	6.46	8.07	9.69	11.30	12.92	14.53	16.15	17.76	19.38	20.99	22.60	24.22	25.83	27.45	29.06	30.68
2	6.67	8.33	10.00	11.67	13.33	15.00	16.67	18.33	20.00	21.67	23.33	25.00	26.67	28.33	30.00	31.67

WEIGHTS OF FLAT ROLLED IRON PER LINEAL FOOT.

(CONTINUED.)

Thickness in Inches.	5″	5¼″	5½″	5¾″	6″	6¼″	6½″	6¾″	7″	7¼″	7½″	7¾″	8″	8¼″	8½″	8¾″
1/16	1.04	1.09	1.15	1.20	1.25	1.30	1.35	1.41	1.46	1.51	1.56	1.61	1.67	1.72	1.77	1.82
1/8	2.08	2.19	2.29	2.40	2.50	2.60	2.71	2.81	2.92	3.02	3.13	3.23	3.33	3.44	3.54	3.65
3/16	3.13	3.28	3.44	3.59	3.75	3.91	4.06	4.22	4.38	4.53	4.69	4.84	5.00	5.16	5.31	5.47
1/4	4.17	4.38	4.58	4.79	5.00	5.21	5.42	5.63	5.83	6.04	6.25	6.46	6.67	6.88	7.08	7.29
5/16	5.21	5.47	5.73	5.99	6.25	6.51	6.77	7.03	7.29	7.55	7.81	8.07	8.33	8.59	8.85	9.11
3/8	6.25	6.56	6.88	7.19	7.50	7.81	8.13	8.44	8.75	9.06	9.38	9.69	10.00	10.31	10.63	10.94
7/16	7.29	7.66	8.02	8.39	8.75	9.11	9.48	9.84	10.21	10.57	10.94	11.30	11.67	12.03	12.40	12.76
1/2	8.33	8.75	9.17	9.58	10.00	10.42	10.83	11.25	11.67	12.08	12.50	12.92	13.33	13.75	14.17	14.58
9/16	9.38	9.84	10.31	10.78	11.25	11.72	12.19	12.66	13.13	13.59	14.06	14.53	15.00	15.47	15.94	16.41
5/8	10.42	10.94	11.46	11.98	12.50	13.02	13.54	14.06	14.58	15.10	15.63	16.15	16.67	17.19	17.71	18.23
11/16	11.46	12.03	12.60	13.18	13.75	14.32	14.90	15.47	16.04	16.61	17.19	17.76	18.33	18.91	19.48	20.05
3/4	12.50	13.13	13.75	14.38	15.00	15.63	16.25	16.88	17.50	18.13	18.75	19.38	20.00	20.63	21.25	21.88
13/16	13.54	14.22	14.90	15.57	16.25	16.93	17.60	18.28	18.96	19.64	20.32	20.99	21.67	22.34	23.02	23.70
7/8	14.58	15.31	16.04	16.77	17.50	18.23	18.96	19.69	20.42	21.15	21.88	22.60	23.33	24.06	24.79	25.52
15/16	15.63	16.41	17.19	17.97	18.75	19.53	20.31	21.09	21.88	22.66	23.44	24.22	25.00	25.78	26.56	27.34
1	16.67	17.50	18.33	19.17	20.00	20.83	21.67	22.50	23.33	24.17	25.00	25.83	26.67	27.50	28.33	29.17
1 1/16	17.71	18.59	19.48	20.36	21.25	22.14	23.02	23.91	24.79	25.68	26.56	27.45	28.33	29.22	30.10	30.99
1 1/8	18.75	19.69	20.63	21.56	22.50	23.44	24.38	25.31	26.25	27.19	28.13	29.06	30.00	30.94	31.88	32.81
1 3/16	19.79	20.78	21.77	22.76	23.75	24.74	25.73	26.72	27.71	28.70	29.69	30.68	31.67	32.66	33.65	34.64
1 1/4	20.83	21.88	22.92	23.96	25.00	26.04	27.08	28.13	29.17	30.21	31.25	32.29	33.33	34.38	35.42	36.46
1 5/16	21.88	22.97	24.06	25.16	26.25	27.34	28.44	29.53	30.62	31.72	32.81	33.91	35.00	36.09	37.19	38.28
1 3/8	22.92	24.06	25.21	26.35	27.50	28.65	29.79	30.94	32.08	33.23	34.38	35.52	36.67	37.81	38.96	40.10
1 7/16	23.96	25.16	26.35	27.55	28.75	29.95	31.15	32.34	33.54	34.74	35.94	37.14	38.33	39.53	40.73	41.93
1 1/2	25.00	26.25	27.50	28.75	30.00	31.25	32.50	33.75	35.00	36.25	37.50	38.75	40.00	41.25	42.50	43.75
1 9/16	26.04	27.34	28.65	29.95	31.25	32.55	33.85	35.16	36.46	37.76	39.06	40.36	41.67	42.97	44.27	45.57
1 5/8	27.08	28.44	29.79	31.15	32.50	33.85	35.21	36.56	37.92	39.27	40.63	41.98	43.33	44.69	46.04	47.40
1 11/16	28.13	29.53	30.94	32.34	33.75	35.16	36.56	37.97	39.38	40.78	42.19	43.59	45.00	46.41	47.81	49.22
1 3/4	29.17	30.63	32.08	33.54	35.00	36.46	37.92	39.38	40.83	42.29	43.75	45.21	46.67	48.13	49.58	51.04
1 13/16	30.21	31.72	33.23	34.74	36.25	37.76	39.27	40.78	42.29	43.80	45.31	46.82	48.33	49.84	51.35	52.86
1 7/8	31.25	32.81	34.38	35.94	37.50	39.06	40.63	42.19	43.75	45.31	46.88	48.44	50.00	51.56	53.13	54.69
1 15/16	32.29	33.91	35.52	37.14	38.75	40.36	41.98	43.59	45.21	46.82	48.44	50.05	51.67	53.28	54.90	56.51
2	33.33	35.00	36.67	38.33	40.00	41.67	43.33	45.00	46.67	48.33	50.00	51.67	53.33	55.00	56.67	58.33

WEIGHTS OF FLAT ROLLED IRON PER LINEAL FOOT.

(CONTINUED.)

Thickness in Inches.	9"	9¼"	9½"	9¾"	10"	10¼"	10½"	10¾"	11"	11¼"	11½"	11¾"	12"	12¼"	12½"	12¾"
1/16	1.88	1.93	1.98	2.03	2.08	2.14	2.19	2.24	2.29	2.34	2.40	2.45	2.50	2.55	2.60	2.66
1/8	3.75	3.85	3.96	4.06	4.17	4.27	4.38	4.48	4.58	4.69	4.79	4.90	5.00	5.10	5.21	5.31
3/16	5.63	5.78	5.94	6.09	6.25	6.41	6.56	6.72	6.88	7.03	7.19	7.34	7.50	7.66	7.81	7.97
1/4	7.50	7.71	7.92	8.13	8.33	8.54	8.75	8.96	9.17	9.38	9.58	9.79	10.00	10.21	10.42	10.63
5/16	9.38	9.64	9.90	10.16	10.42	10.68	10.94	11.20	11.46	11.72	11.98	12.24	12.50	12.76	13.02	13.28
3/8	11.25	11.56	11.88	12.19	12.50	12.81	13.13	13.44	13.75	14.06	14.38	14.69	15.00	15.31	15.63	15.94
7/16	13.13	13.49	13.85	14.22	14.58	14.95	15.31	15.68	16.04	16.41	16.77	17.14	17.50	17.86	18.23	18.59
1/2	15.00	15.42	15.83	16.25	16.67	17.08	17.50	17.92	18.33	18.75	19.17	19.58	20.00	20.42	20.83	21.25
9/16	16.88	17.34	17.81	18.28	18.75	19.22	19.69	20.16	20.63	21.09	21.56	22.03	22.50	22.97	23.44	23.91
5/8	18.75	19.27	19.79	20.31	20.83	21.35	21.88	22.40	22.92	23.44	23.96	24.48	25.00	25.52	26.04	26.56
11/16	20.63	21.20	21.77	22.34	22.92	23.49	24.06	24.64	25.21	25.78	26.35	26.93	27.50	28.07	28.65	29.22
3/4	22.50	23.13	23.75	24.38	25.00	25.62	26.25	26.88	27.50	28.13	28.75	29.38	30.00	30.63	31.25	31.88
13/16	24.38	25.05	25.73	26.41	27.08	27.76	28.44	29.11	29.79	30.47	31.15	31.82	32.50	33.18	33.85	34.53
7/8	26.25	26.98	27.71	28.44	29.17	29.90	30.63	31.35	32.08	32.81	33.54	34.27	35.00	35.73	36.46	37.19
15/16	28.13	28.91	29.69	30.47	31.25	32.03	32.81	33.59	34.38	35.16	35.94	36.72	37.50	38.28	39.06	39.84
1	30.00	30.83	31.67	32.50	33.33	34.17	35.00	35.83	36.67	37.50	38.33	39.17	40.00	40.83	41.67	42.50
1 1/16	31.88	32.76	33.65	34.53	35.42	36.30	37.19	38.07	38.96	39.84	40.73	41.61	42.50	43.39	44.27	45.16
1 1/8	33.75	34.69	35.63	36.56	37.50	38.44	39.38	40.31	41.25	42.19	43.13	44.06	45.00	45.94	46.88	47.81
1 3/16	35.63	36.61	37.60	38.59	35.58	40.57	41.56	42.55	43.54	44.53	45.52	46.51	47.50	48.49	49.48	50.47
1 1/4	37.50	38.54	39.58	40.63	41.67	42.71	43.75	44.79	45.83	46.88	47.92	48.96	50.00	51.04	52.08	53.13
1 5/16	39.38	40.47	41.56	42.66	43.75	44.84	45.94	47.03	48.13	49.22	50.31	51.41	52.50	53.59	54.69	55.78
1 3/8	41.25	42.40	43.54	44.69	45.83	46.98	48.13	49.27	50.42	51.56	52.71	53.85	55.00	56.15	57.29	58.44
1 7/16	43.13	44.32	45.52	46.72	47.92	49.11	50.31	51.51	52.71	53.91	55.10	56.30	57.50	58.70	59.90	61.09
1 1/2	45.00	46.25	47.50	48.75	50.00	51.25	52.50	53.75	55.00	56.25	57.50	58.75	60.00	61.25	62.50	63.75
1 9/16	46.88	48.18	49.48	50.78	52.08	53.39	54.69	55.99	57.29	58.59	59.90	61.20	62.50	63.80	65.10	66.41
1 5/8	48.75	50.10	51.46	52.81	54.17	55.52	56.88	58.23	59.58	60.94	62.29	63.65	65.00	66.35	67.71	69.06
1 11/16	50.63	52.03	53.44	54.84	56.25	57.66	59.06	60.47	61.88	63.28	64.69	66.09	67.50	68.91	70.31	71.72
1 3/4	52.50	53.96	55.42	56.88	58.33	59.79	61.25	62.71	64.17	65.63	67.08	68.54	70.00	71.46	72.92	74.38
1 13/16	54.38	55.89	57.40	58.91	60.42	61.93	63.44	64.95	66.46	67.97	69.48	70.99	72.50	74.01	75.52	77.03
1 7/8	56.25	57.81	59.38	60.94	62.50	64.06	65.63	67.19	68.75	70.31	71.88	73.44	75.00	76.56	78.13	79.69
1 15/16	58.13	59.74	61.35	62.97	64.58	66.20	67.81	69.43	71.04	72.66	74.27	75.89	77.50	79.11	80.73	82.34
2	60.00	61.67	63.33	65.00	66.67	68.33	70.00	71.67	73.33	75.00	76.67	78.33	80.00	81.67	83.33	85.00

AREAS OF FLAT ROLLED IRON,

For Thicknesses from 1/16 in. to 2 in. and Widths from 1 in. to 12¾ in.

Thickness in Inches.	1"	1¼"	1½"	1¾"	2"	2¼"	2½"	2¾"	3"	3¼"	3½"	3¾"	4"	4¼"	4½"	4¾"
1/16	.063	.078	.094	.109	.125	.141	.156	.172	.188	.203	.219	.234	.250	.266	.281	.297
1/8	.125	.156	.188	.219	.250	.281	.313	.344	.375	.406	.438	.469	.500	.531	.563	.594
3/16	.188	.234	.281	.328	.375	.422	.469	.516	.563	.609	.656	.703	.750	.797	.844	.891
1/4	.250	.313	.375	.438	.500	.563	.625	.688	.750	.813	.875	.938	1.00	1.06	1.13	1.19
5/16	.313	.391	.469	.547	.625	.703	.781	.859	.938	1.02	1.09	1.17	1.25	1.33	1.41	1.48
3/8	.375	.469	.563	.656	.750	.844	.938	1.03	1.13	1.22	1.31	1.41	1.50	1.59	1.69	1.78
7/16	.438	.547	.656	.766	.875	.984	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86	1.97	2.08
1/2	.500	.625	.750	.875	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13	2.25	2.38
9/16	.563	.703	.844	.984	1.13	1.27	1.41	1.55	1.69	1.83	1.97	2.11	2.25	2.39	2.53	2.67
5/8	.625	.781	.938	1.09	1.25	1.41	1.56	1.72	1.88	2.03	2.19	2.34	2.50	2.66	2.81	2.97
11/16	.688	.859	1.03	1.20	1.38	1.55	1.72	1.89	2.06	2.23	2.41	2.58	2.75	2.92	3.09	3.27
3/4	.750	.938	1.13	1.31	1.50	1.69	1.88	2.06	2.25	2.44	2.63	2.81	3.00	3.19	3.38	3.56
13/16	.813	1.02	1.22	1.42	1.63	1.83	2.03	2.23	2.44	2.64	2.84	3.05	3.25	3.45	3.66	3.86
7/8	.875	1.09	1.31	1.53	1.75	1.97	2.19	2.41	2.63	2.84	3.06	3.28	3.50	3.72	3.94	4.16
15/16	.938	1.17	1.41	1.64	1.88	2.11	2.34	2.58	2.81	3.05	3.28	3.52	3.75	3.98	4.22	4.45
1	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75
1 1/16	1.06	1.33	1.59	1.86	2.13	2.39	2.66	2.92	3.19	3.45	3.72	3.98	4.25	4.52	4.78	5.05
1 1/8	1.13	1.41	1.69	1.97	2.25	2.53	2.81	3.09	3.38	3.66	3.94	4.22	4.50	4.78	5.06	5.34
1 3/16	1.19	1.48	1.78	2.08	2.38	2.67	2.97	3.27	3.56	3.86	4.16	4.45	4.75	5.05	5.34	5.64
1 1/4	1.25	1.56	1.88	2.19	2.50	2.81	3.13	3.44	3.75	4.06	4.38	4.69	5.00	5.31	5.63	5.94
1 5/16	1.31	1.64	1.97	2.30	2.63	2.95	3.28	3.61	3.94	4.27	4.59	4.92	5.25	5.58	5.91	6.23
1 3/8	1.38	1.72	2.06	2.41	2.75	3.09	3.44	3.78	4.13	4.47	4.81	5.16	5.50	5.84	6.19	6.53
1 7/16	1.44	1.80	2.16	2.52	2.88	3.23	3.59	3.95	4.31	4.67	5.03	5.39	5.75	6.11	6.47	6.83
1 1/2	1.50	1.88	2.25	2.63	3.00	3.38	3.75	4.13	4.50	4.88	5.25	5.63	6.00	6.38	6.75	7.13
1 9/16	1.56	1.95	2.34	2.73	3.13	3.52	3.91	4.30	4.69	5.08	5.47	5.86	6.25	6.64	7.03	7.42
1 5/8	1.63	2.03	2.44	2.84	3.25	3.66	4.06	4.47	4.88	5.28	5.69	6.09	6.50	6.91	7.31	7.72
1 11/16	1.69	2.11	2.53	2.95	3.38	3.80	4.22	4.64	5.06	5.48	5.91	6.33	6.75	7.17	7.59	8.02
1 3/4	1.75	2.19	2.63	3.06	3.50	3.94	4.38	4.81	5.25	5.69	6.13	6.56	7.00	7.44	7.88	8.31
1 13/16	1.81	2.27	2.72	3.17	3.63	4.08	4.53	4.98	5.44	5.89	6.34	6.80	7.25	7.70	8.16	8.61
1 7/8	1.88	2.34	2.81	3.28	3.75	4.22	4.69	5.16	5.63	6.09	6.56	7.03	7.50	7.97	8.44	8.91
1 15/16	1.94	2.42	2.91	3.39	3.88	4.36	4.84	5.33	5.81	6.30	6.78	7.27	7.75	8.23	8.72	9.20
2	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50

AREAS OF FLAT ROLLED IRON.

(CONTINUED)

Thickness in Inches.	5"	5¼"	5½"	5¾"	6"	6¼"	6½"	6¾"	7"	7¼"	7½"	7¾"	8"	8¼"	8½"	8¾"
1/16	.313	.328	.344	.359	.375	.391	.406	.422	.438	.453	.469	.484	.500	.516	.531	.547
1/8	.625	.656	.688	.719	.750	.781	.813	.844	.875	.906	.938	.969	1.00	1.03	1.06	1.09
3/16	.938	.984	1.03	1.08	1.13	1.17	1.22	1.27	1.31	1.36	1.41	1.45	1.50	1.55	1.59	1.64
1/4	1.25	1.31	1.38	1.44	1.50	1.56	1.63	1.69	1.75	1.81	1.88	1.94	2.00	2.06	2.13	2.19
5/16	1.56	1.64	1.72	1.80	1.88	1.95	2.03	2.11	2.19	2.27	2.34	2.42	2.50	2.58	2.66	2.73
3/8	1.88	1.97	2.06	2.16	2.25	2.34	2.44	2.53	2.63	2.72	2.81	2.91	3.00	3.09	3.19	3.28
7/16	2.19	2.30	2.41	2.52	2.63	2.73	2.84	2.95	3.06	3.17	3.28	3.39	3.50	3.61	3.72	3.83
1/2	2.50	2.63	2.75	2.88	3.00	3.13	3.25	3.38	3.50	3.63	3.75	3.88	4.00	4.13	4.25	4.38
9/16	2.81	2.95	3.09	3.23	3.38	3.52	3.66	3.80	3.94	4.08	4.22	4.36	4.50	4.64	4.78	4.92
5/8	3.13	3.28	3.44	3.59	3.75	3.91	4.06	4.22	4.38	4.53	4.69	4.84	5.00	5.16	5.31	5.47
11/16	3.44	3.61	3.78	3.95	4.13	4.30	4.47	4.64	4.81	4.98	5.16	5.33	5.50	5.67	5.84	6.02
3/4	3.75	3.94	4.13	4.31	4.50	4.69	4.88	5.06	5.25	5.44	5.63	5.81	6.00	6.19	6.38	6.56
13/16	4.06	4.27	4.47	4.67	4.88	5.08	5.28	5.48	5.69	5.89	6.09	6.30	6.50	6.70	6.91	7.11
7/8	4.38	4.59	4.81	5.03	5.25	5.47	5.69	5.91	6.13	6.34	6.56	6.78	7.00	7.22	7.44	7.66
15/16	4.69	4.92	5.16	5.39	5.63	5.86	6.09	6.33	6.56	6.80	7.03	7.27	7.50	7.73	7.97	8.20
1	5.00	5.25	5.50	5.75	6.00	6.25	6.50	6.75	7.00	7.25	7.50	7.75	8.00	8.25	8.50	8.75
1 1/16	5.31	5.58	5.84	6.11	6.38	6.64	6.91	7.17	7.44	7.70	7.97	8.23	8.50	8.77	9.03	9.30
1 1/8	5.63	5.91	6.19	6.47	6.75	7.03	7.31	7.59	7.88	8.16	8.44	8.72	9.00	9.28	9.56	9.84
1 3/16	5.94	6.23	6.53	6.83	7.13	7.42	7.72	8.02	8.31	8.61	8.91	9.20	9.50	9.80	10.09	10.39
1 1/4	6.25	6.56	6.88	7.19	7.50	7.81	8.13	8.44	8.75	9.06	9.38	9.69	10.00	10.31	10.63	10.94
1 5/16	6.56	6.89	7.22	7.55	7.88	8.20	8.53	8.86	9.19	9.52	9.84	10.17	10.50	10.83	11.16	11.48
1 3/8	6.88	7.22	7.56	7.91	8.25	8.59	8.94	9.28	9.63	9.97	10.31	10.66	11.00	11.34	11.69	12.03
1 7/16	7.19	7.55	7.91	8.27	8.63	8.98	9.34	9.70	10.06	10.42	10.78	11.14	11.50	11.86	12.22	12.58
1 1/2	7.50	7.88	8.25	8.63	9.00	9.38	9.75	10.13	10.50	10.88	11.25	11.63	12.00	12.38	12.75	13.13
1 9/16	7.81	8.20	8.59	8.98	9.38	9.77	10.16	10.55	10.94	11.33	11.72	12.11	12.50	12.89	13.28	13.67
1 5/8	8.13	8.53	8.94	9.34	9.75	10.16	10.56	10.97	11.38	11.78	12.19	12.59	13.00	13.41	13.81	14.22
1 11/16	8.44	8.86	9.28	9.70	10.13	10.55	10.97	11.39	11.81	12.23	12.66	13.08	13.50	13.92	14.34	14.77
1 3/4	8.75	9.19	9.63	10.06	10.50	10.94	11.38	11.81	12.25	12.69	13.13	13.56	14.00	14.44	14.88	15.31
1 13/16	9.06	9.52	9.97	10.42	10.88	11.33	11.78	12.23	12.69	13.14	13.59	14.05	14.50	14.95	15.41	15.86
1 7/8	9.38	9.84	10.31	10.78	11.25	11.72	12.19	12.66	13.13	13.59	14.06	14.53	15.00	15.47	15.94	16.41
1 15/16	9.69	10.17	10.66	11.14	11.63	12.11	12.59	13.08	13.56	14.05	14.53	15.02	15.50	15.98	16.47	16.95
2	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00	15.50	16.00	16.50	17.00	17.50

AREAS OF FLAT ROLLED IRON.

(CONTINUED.)

Thickness in Inches.	9"	9¼"	9½"	9¾"	10"	10¼"	10½"	10¾"	11"	11¼"	11½"	11¾"	12"	12¼"	12½"	12¾"
1/16	.563	.578	.594	.609	.625	.641	.656	.672	.688	.703	.719	.734	.750	.766	.781	.797
1/8	1.13	1.16	1.19	1.22	1.25	1.28	1.31	1.34	1.38	1.41	1.44	1.47	1.50	1.53	1.56	1.59
3/16	1.69	1.73	1.78	1.83	1.88	1.92	1.97	2.02	2.06	2.11	2.16	2.20	2.25	2.30	2.34	2.39
1/4	2.25	2.31	2.38	2.44	2.50	2.56	2.63	2.69	2.75	2.81	2.88	2.94	3.00	3.06	3.13	3.19
5/16	2.81	2.89	2.97	3.05	3.13	3.20	3.28	3.36	3.44	3.52	3.59	3.67	3.75	3.83	3.91	3.98
3/8	3.38	3.47	3.56	3.66	3.75	3.84	3.94	4.03	4.13	4.22	4.31	4.41	4.50	4.59	4.69	4.78
7/16	3.94	4.05	4.16	4.27	4.38	4.48	4.59	4.70	4.81	4.92	5.03	5.14	5.25	5.36	5.47	5.58
1/2	4.50	4.63	4.75	4.88	5.00	5.13	5.25	5.38	5.50	5.63	5.75	5.88	6.00	6.13	6.25	6.38
9/16	5.06	5.20	5.34	5.48	5.63	5.77	5.91	6.05	6.19	6.33	6.47	6.61	6.75	6.89	7.03	7.17
5/8	5.63	5.78	5.94	6.09	6.25	6.41	6.56	6.72	6.88	7.03	7.19	7.34	7.50	7.66	7.81	7.97
11/16	6.19	6.36	6.53	6.70	6.88	7.05	7.22	7.39	7.56	7.73	7.91	8.08	8.25	8.42	8.59	8.77
3/4	6.75	6.94	7.13	7.31	7.50	7.69	7.88	8.06	8.25	8.44	8.63	8.81	9.00	9.19	9.38	9.56
13/16	7.31	7.52	7.72	7.92	8.13	8.33	8.53	8.73	8.94	9.14	9.34	9.55	9.75	9.95	10.16	10.36
7/8	7.88	8.09	8.31	8.53	8.75	8.97	9.19	9.41	9.63	9.84	10.06	10.28	10.50	10.72	10.94	11.16
15/16	8.44	8.67	8.91	9.14	9.38	9.61	9.84	10.08	10.31	10.55	10.78	11.02	11.25	11.48	11.72	11.95
1	9.00	9.25	9.50	9.75	10.00	10.25	10.50	10.75	11.00	11.25	11.50	11.75	12.00	12.25	12.50	12.75
1 1/16	9.56	9.83	10.09	10.36	10.63	10.89	11.16	11.42	11.69	11.95	12.22	12.48	12.75	13.02	13.28	13.55
1 1/8	10.13	10.41	10.69	10.97	11.25	11.53	11.81	12.09	12.38	12.66	12.94	13.22	13.50	13.78	14.06	14.34
1 3/16	10.69	10.98	11.28	11.58	11.88	12.17	12.47	12.77	13.06	13.36	13.66	13.95	14.25	14.55	14.84	15.14
1 1/4	11.25	11.56	11.88	12.19	12.50	12.81	13.13	13.44	13.75	14.06	14.38	14.69	15.00	15.31	15.63	15.94
1 5/16	11.81	12.14	12.47	12.80	13.13	13.45	13.78	14.11	14.44	14.77	15.09	15.42	15.75	16.08	16.41	16.73
1 3/8	12.38	12.72	13.06	13.41	13.75	14.09	14.44	14.78	15.13	15.47	15.81	16.16	16.50	16.84	17.19	17.53
1 7/16	12.94	13.30	13.66	14.02	14.38	14.73	15.09	15.45	15.81	16.17	16.53	16.89	17.25	17.61	17.97	18.33
1 1/2	13.50	13.88	14.25	14.63	15.00	15.38	15.75	16.13	16.50	16.88	17.25	17.63	18.00	18.38	18.75	19.13
1 9/16	14.06	14.45	14.84	15.23	15.63	16.02	16.41	16.80	17.19	17.58	17.97	18.36	18.75	19.14	19.53	19.92
1 5/8	14.63	15.03	15.44	15.84	16.25	16.66	17.06	17.47	17.88	18.28	18.69	19.09	19.50	19.91	20.31	20.72
1 11/16	15.19	15.61	16.03	16.45	16.88	17.30	17.72	18.14	18.56	18.98	19.41	19.83	20.25	20.67	21.09	21.52
1 3/4	15.75	16.19	16.63	17.06	17.50	17.94	18.38	18.81	19.25	19.69	20.13	20.56	21.00	21.44	21.88	22.31
1 13/16	16.31	16.77	17.22	17.67	18.13	18.58	19.03	19.48	19.94	20.39	20.84	21.30	21.75	22.20	22.66	23.11
1 7/8	16.88	17.34	17.81	18.28	18.75	19.22	19.69	20.16	20.63	21.09	21.56	22.03	22.50	22.97	23.44	23.91
1 15/16	17.44	17.92	18.41	18.89	19.38	19.86	20.34	20.83	21.31	21.80	22.28	22.77	23.25	23.73	24.22	24.70
2	18.00	18.50	19.00	19.50	20.00	20.50	21.00	21.50	22.00	22.50	23.00	23.50	24.00	24.50	25.00	25.50

WEIGHTS AND AREAS OF SQUARE AND ROUND BARS OF WROUGHT IRON.

One cubic foot weighing 480 lbs.

Thickness or Diameter in Inches.	Weight of □ Bar One Ft. long.	Weight of ○ Bar One Ft. long.	Area of □ Bar in sq. inches.	Area of ○ Bar in sq. inches.	Thickness or Diameter in Inches.	Weight of □ Bar One Ft. long.	Weight of ○ Bar One Ft. long.	Area of □ Bar in sq. inches.	Area of ○ Bar in sq. inches.
0					2	13.33	10.47	4.0000	3.1416
1/16	.013	.010	.0039	.0031	1/16	14.18	11.14	4.2539	3.3410
1/8	.052	.041	.0156	.0123	1/8	15.05	11.82	4.5156	3.5466
3/16	.117	.092	.0352	.0276	3/16	15.95	12.53	4.7852	3.7583
1/4	.208	.164	.0625	.0491	1/4	16.88	13.25	5.0625	3.9761
5/16	.326	.256	.0977	.0767	5/16	17.83	14.00	5.3477	4.2000
3/8	.469	.368	.1406	.1104	3/8	18.80	14.77	5.6406	4.4301
7/16	.638	.501	.1914	.1503	7/16	19.80	15.55	5.9414	4.6664
1/2	.833	.654	.2500	.1963	1/2	20.83	16.36	6.2500	4.9087
9/16	1.055	.828	.3164	.2485	9/16	21.89	17.19	6.5664	5.1572
5/8	1.302	1.023	.3906	.3068	5/8	22.97	18.04	6.8906	5.4119
11/16	1.576	1.237	.4727	.3712	11/16	24.08	18.91	7.2227	5.6727
3/4	1.875	1.473	.5625	.4418	3/4	25.21	19.80	7.5625	5.9396
13/16	2.201	1.728	.6602	.5185	13/16	26.37	20.71	7.9102	6.2126
7/8	2.552	2.004	.7656	.6013	7/8	27.55	21.64	8.2656	6.4918
15/16	2.930	2.301	.8789	.6903	15/16	28.76	22.59	8.6289	6.7771
1	3.333	2.618	1.0000	.7854	3	30.00	23.56	9.0000	7.0686
1/16	3.763	2.955	1.1289	.8866	1/16	31.26	24.55	9.3789	7.3662
1/8	4.219	3.313	1.2656	.9940	1/8	32.55	25.57	9.7656	7.6699
3/16	4.701	3.692	1.4102	1.1075	3/16	33.87	26.60	10.160	7.9798
1/4	5.208	4.091	1.5625	1.2272	1/4	35.21	27.65	10.563	8.2958
5/16	5.742	4.510	1.7227	1.3530	5/16	36.58	28.73	10.973	8.6179
3/8	6.302	4.950	1.8906	1.4849	3/8	37.97	29.82	11.391	8.9462
7/16	6.888	5.410	2.0664	1.6230	7/16	39.39	30.94	11.816	9.2806
1/2	7.500	5.890	2.2500	1.7671	1/2	40.83	32.07	12.250	9.6211
9/16	8.138	6.392	2.4414	1.9175	9/16	42.30	33.23	12.691	9.9678
5/8	8.802	6.913	2.6406	2.0739	5/8	43.80	34.40	13.141	10.321
11/16	9.492	7.455	2.8477	2.2365	11/16	45.33	35.60	13.598	10.680
3/4	10.21	8.018	3.0625	2.4053	3/4	46.88	36.82	14.063	11.045
13/16	10.95	8.601	3.2852	2.5802	13/16	48.45	38.05	14.535	11.416
7/8	11.72	9.204	3.5156	2.7612	7/8	50.05	39.31	15.016	11.793
15/16	12.51	9.828	3.7539	2.9483	15/16	51.68	40.59	15.504	12.177

WEIGHTS AND AREAS OF SQUARE AND ROUND BARS OF WROUGHT IRON.

One cubic foot weighing 480 lbs. (CONTINUED.)

Thickness or Diameter in Inches.	Weight of □ Bar One Ft. long.	Weight of ○ Bar One Ft. long.	Area of □ Bar in sq. inches.	Area of ○ Bar in sq. inches.	Thickness or Diameter in Inches.	Weight of □ Bar One Ft. long.	Weight of ○ Bar One Ft. long.	Area of □ Bar in sq. inches.	Area of ○ Bar in sq. inches.
4	53.33	41.89	16.000	12.566	6	120.0	94.25	36.000	28.274
$\frac{1}{16}$	55.01	43.21	16.504	12.962	$\frac{1}{16}$	122.5	96.22	36.754	28.866
$\frac{1}{8}$	56.72	44.55	17.016	13.364	$\frac{1}{8}$	125.1	98.22	37.516	29.465
$\frac{3}{16}$	58.45	45.91	17.535	13.772	$\frac{3}{16}$	127.6	100.2	38.285	30.069
$\frac{1}{4}$	60.21	47.29	18.063	14.186	$\frac{1}{4}$	130.2	102.3	39.063	30.680
$\frac{5}{16}$	61.99	48.69	18.598	14.607	$\frac{5}{16}$	132.8	104.3	39.848	31.296
$\frac{3}{8}$	63.80	50.11	19.141	15.033	$\frac{3}{8}$	135.5	106.4	40.641	31.919
$\frac{7}{16}$	65.64	51.55	19.691	15.466	$\frac{7}{16}$	138.1	108.5	41.441	32.548
$\frac{1}{2}$	67.50	53.01	20.250	15.904	$\frac{1}{2}$	140.8	110.6	42.250	33.183
$\frac{9}{16}$	69.39	54.50	20.816	16.349	$\frac{9}{16}$	143.6	112.7	43.066	33.824
$\frac{5}{8}$	71.30	56.00	21.391	16.800	$\frac{5}{8}$	146.3	114.9	43.891	34.472
$\frac{11}{16}$	73.24	57.52	21.973	17.257	$\frac{11}{16}$	149.1	117.1	44.723	35.125
$\frac{3}{4}$	75.21	59.07	22.563	17.721	$\frac{3}{4}$	151.9	119.3	45.563	35.785
$\frac{13}{16}$	77.20	60.63	23.160	18.190	$\frac{13}{16}$	154.7	121.5	46.410	36.450
$\frac{7}{8}$	79.22	62.22	23.766	18.665	$\frac{7}{8}$	157.6	123.7	47.266	37.122
$\frac{15}{16}$	81.26	63.82	24.379	19.147	$\frac{15}{16}$	160.4	126.0	48.129	37.800
5	83.33	65.45	25.000	19.635	7	163.3	128.3	49.000	38.485
$\frac{1}{16}$	85.43	67.10	25.629	20.129	$\frac{1}{16}$	166.3	130.6	49.879	39.175
$\frac{1}{8}$	87.55	68.76	26.266	20.629	$\frac{1}{8}$	169.2	132.9	50.766	39.871
$\frac{3}{16}$	89.70	70.45	26.910	21.135	$\frac{3}{16}$	172.2	135.2	51.660	40.574
$\frac{1}{4}$	91.88	72.16	27.563	21.648	$\frac{1}{4}$	175.2	137.6	52.563	41.282
$\frac{5}{16}$	94.08	73.89	28.223	22.166	$\frac{5}{16}$	178.2	140.0	53.473	41.997
$\frac{3}{8}$	96.30	75.64	28.891	22.691	$\frac{3}{8}$	181.3	142.4	54.391	42.718
$\frac{7}{16}$	98.55	77.40	29.566	23.221	$\frac{7}{16}$	184.4	144.8	55.316	43.445
$\frac{1}{2}$	100.8	79.19	30.250	23.758	$\frac{1}{2}$	187.5	147.3	56.250	44.179
$\frac{9}{16}$	103.1	81.00	30.941	24.301	$\frac{9}{16}$	190.6	149.7	57.191	44.918
$\frac{5}{8}$	105.5	82.83	31.641	24.850	$\frac{5}{8}$	193.8	152.2	58.141	45.664
$\frac{11}{16}$	107.8	84.69	32.348	25.406	$\frac{11}{16}$	197.0	154.7	59.098	46.415
$\frac{3}{4}$	110.2	86.56	33.063	25.967	$\frac{3}{4}$	200.2	157.2	60.063	47.173
$\frac{13}{16}$	112.6	88.45	33.785	26.535	$\frac{13}{16}$	203.5	159.8	61.035	47.937
$\frac{7}{8}$	115.1	90.36	34.516	27.109	$\frac{7}{8}$	206.7	162.4	62.016	48.707
$\frac{15}{16}$	117.5	92.29	35.254	27.688	$\frac{15}{16}$	210.0	164.9	63.004	49.483

In answer to a letter concerning their limiting lengths of sections, Messrs. Carnegie Bros. & Co. wrote as follows on Mar. 31st '84.

"We have your card of the 6th February. Beams and channels can be rolled to 50, 60 and 70 feet lengths, depending somewhat upon the size, and the same may be said of angles and plates.

We roll in plates nothing exceeding 36″ in width. In this country beams and channels are sold at 3 1—2 cts. per lb., on cars Pittsburg, for lengths not exceeding 30 feet. For lengths above this there is an additional charge of 1—4 cent per lb. for each 5 feet or fraction thereof. For export, however, we could waive the additional charge for lengths 40 feet and under.

Angles and plates are sold at 2 4—10 cts. per lb., on cars here, with an additional charge as above for lengths over 50 feet." *

Although this company does not roll plates wider than thirty-six inches, other American companies roll them as wide as six feet or even wider. Such plates would not be longer than fifteen feet and not over half an inch thick.

* The latest quotations (Mar. '85) are about ½ cent per pound less than the above.

CHAPTER III.

LIST OF MEMBERS.

In the following list of members will be found the names of all the parts both of wood and iron in the bridges with which this treatise deals. Its use will be explained in a subsequent chapter.

List of the Different Members in a Wrought Iron
Pratt or Whipple Truss Railroad Bridge.

MAIN PORTIONS.

Channels.
- Top Chords.
- Batter Braces.
- Posts.
- Upper Lateral Struts.
- Lower Lateral Struts.
- Upper Portal Struts.
- Lower Portal Struts.
- Intermediate Struts.
- Bottom Chord Struts.
- Hip Verticals in Pony Trusses.
- Side Braces.

I Beams.
- Top Chords.
- Batter Braces.
- Bottom Chords.
- Intermediate Struts.
- End Lower Lateral Struts.
- Track Stringers.

Angles.
- Top Chord Built Channels.
- Batter Brace Built Channels.
- Post Built Channels.
- Side Braces.
- Floor Beams.
- Track Stringers.
- Guard Rails. *

Tees.
- Side Braces.

Plate.
- Top Chord Upper Plates.
- Batter Brace Upper Plates.
- Webs of Built Channels.
- Webs of Floor Beams.
- Webs of Track Stringers.
- Flanges of Floor Beams.
- Flanges of Track Stringers.

Flats.
- Main Diagonals.
- Hip Verticals.
- Chord Bars.

Rounds and Squares.
- Counters.
- Hip Verticals.
- Upper Lateral Rods.
- Lower Lateral Rods.
- Portal Vibration Rods.
- Intermediate Vibration Rods.

* Weight included in that of Floor System Proper.

DETAILS.

- Plates.
  - Stay Plates.
    - Top Chords.
    - Batter Braces.
    - Ends of Posts.
    - Middle of Posts.
    - Upper Lateral Struts.
    - Lower Lateral Struts.
    - Upper Portal Struts.
    - Lower Portal Struts.
    - Intermediate Struts.
    - Bottom Chord Struts.
  - Reinforcing and Connecting Plates.
    - Hip Inside.
    - Hip Outside.
    - Top Chord Intermediate Panel Points Inside.
    - Top Chord Intermediate Panel Points Outside.
    - Pin Holes in Bottom Chord Struts.
    - Pin Holes in Shoes.
    - Lower Ends of Posts Inside.
    - Lower Ends of Posts Outside.
    - Middle of Posts at Pin Holes.
    - Reinforcing Plates on Side Braces.
    - Intermediate Struts to Posts.
    - Intermediate Struts to Brackets.
    - Lower Portal Struts to Brackets.
    - Upper Portal Struts to Name Plates.
    - Side Bracing to Floor Beams.
    - Side Bracing to Chord Pins.
    - Track Stringers over Floor Beams.
    - I Beam Track Stringers to Floor Beams.
    - Octagonal Connecting Plates between Track Stringers and Floor Beams.
  - Cover Plates.
    - Hip Joints.
    - Intermediate Panel Points of Top Chords.
  - Filling Plates.
    - At Hips.
    - At Intermediate Panel Points of Top Chords.
    - At Joints in Bottom Chord Struts.
    - Over End Floor Beams.
    - Between Pedestals and Lateral Struts.
  - Jaw Plates.
    - Upper Lateral Struts.
    - Lower Lateral Struts.
    - Upper Portal Struts.
    - Lower Portal Struts.
    - Intermediate Struts.
  - Extension Plates at Upper Ends of Posts.
  - Shoe or Pedestal Plates.
  - Roller Plates.
  - Bed Plates at Pedestals.
  - Bed Plates for Track Stringers including Anchor Plates.
  - Beam Hanger Plates.
  - Name Plates.

Latticing or Lacing.	**Top Chords.** **Batter Braces.** **Bottom Chord Struts.** **Posts.** **Upper Lateral Struts.** **Lower Lateral Struts.** **Upper Portal Struts.** **Lower Portal Struts.** **Intermediate Struts.** **Hip Verticals in Pony Trusses.**
Trussing.	**Hip Verticals in Pony Trusses.** **Lower Chord Bars.**
Pins.	**Top Chords.** **Bottom Chords.** **Middle of Posts.** **Upper Lateral Rod Connection to Jaws and Hip Pins.** **Lower Lateral Rod Connection to Jaws.** **Upper Portal Vibration Pins.** **Lower Portal Vibration Pins.** **Vibration Rod Connection to Lateral Struts.**
Bolts.	**Name Plate Bolts.** **Vibration Rod Connection to Upper Lateral Struts.** **Vibration Rod Connection to Intermediate Struts.** **Portal Struts to Batter Braces.** **Bed Plates to Piers (Anchor Bolts).** **Shoes to Bed Plates.** **Shims to Track Stringers.** * **Ties to Shims (Drift Bolts).** * **Guard Rails to Ties.** * **Temporary Bolts used in Erection.** †
Brackets.	**Lower Portal Struts to Batter Braces.** **Upper Lateral Struts to Posts.** **Intermediate Struts to Posts.** **Stringers to Floor Beams.** **Floor Beams to Top Chords in Deck Bridges.**

* Weight included in that of Floor System Proper.

† Weight not to be included in dead load.

Ornamental Work.
Beam Hangers.
Expansion Rollers.
Roller Frames.
Fillers for Pins.
Tee Iron Fillers over End Floor Beams.
Shoe Pin Supporting Pieces.
Turn-buckles. *
Sleeve-nuts. *
Angles for sides of Roller Plates.
Anchor Pieces for sides of Roller Plates.
Stringer Bracing Frames.
Rails and their Connections. †

Spikes.
- Rails to Ties. †
- Foot Planks to Ties. †

Washers.
- On Shim Bolts.
- On Guard Rail Bolts. †

Nuts.
- On Pins.
- On Bolts.
- On Beam Hangers.
- Lock Nuts.
- Pilot Nuts. ‡

Rivet Heads.
- Upper Plates to Chord Channels.
- Upper Plates to Batter Brace Channels.
- Latticing and Lacing to Channels.
- Latticing to Latticing.
- The various Stay Plates to Channels.
- The various Reinforcing Plates to Channels &c.
- The various Connecting Plates to the parts which they connect.
- Cover Plates to Channels.
- Extension Plates to Channels.
- Upper Plates of Batter Braces to Shoe Plates.
- Batter Brace Connections to Shoe Plates.
- Trussing to parts trussed.
- Ornamental Work Connection.
- Brackets to the parts which they connect.
- Jaw Plates to struts
- Jaw Plates to Jaw Plates.
- Side Braces to Top Chords.
- Angle Irons to Roller Plates.
- Bracing Frames to Stringers.
- Track Stringers to Floor Beam Connecting Plates.
- Post Reinforcing Plates to Floor Beams.

---

* Weight included in that of Rods.
† Weight included in that of Floor System Proper.
‡ Weight not to be included in dead load.

Components of Built Beams.	Web. Top Plate. Bottom Plate. Upper Flange Angles. Lower Flange Angles. Stiffening Angles. Filling Plates. Stringer Supports. Rivet Heads.
Components of Built Track Stringers.	Web. Bottom Plate. Upper Flange Angles. Lower Flange Angles. Stiffening Angles. Filling Plates. Connecting Plates to Floor Beams. Splice Plates for Web. Splice Plates for Flanges. Rivet Heads.
Components of Rolled Track Stringers	I Beam. Stiffening Angles at Supports. Connecting Plates to Floor Beams. Rivet Heads.

LUMBER.

Shims.
Ties.
Foot Plank.

N. B. The *details* of floor beams and built track stringers do not appear among the other details in the list, although the webs and flange angles do appear in list of *main portions*. Rails and their splice plates do not appear in the list, but their weight is included in that of the Floor System Proper (see Chapter V).

LIST OF MEMBERS IN A PLATE GIRDER SPAN.

Webs.
Top Plates.
Bottom Plates.
Upper Flange Angles
Lower Flange Angles.
Vertical Stiffening Angles.
Inclined Stiffening Angles.
Filling Plates.
Shoe Plates.
Bed Plates with Anchor Plates.
Anchor Bolts with their Nuts.
Bracing Frames at ends.
Diagonal Bracing Angles.
Connecting Plates for same.
Rivet Heads.
Tie Bolts.
Spikes for Rails.
Guard Rail Angles.
Washers for Tie Bolts.

CHAPTER IV.

GENERAL DESCRIPTION AND REMARKS.

If in the last chapter or any other part of the book the reader encounter technical terms with which he is not familiar, he can ascertain their meaning by turning to the Glossary and, if necessary, consulting the plate or plates there referred to; and he is strongly advised to thoroughly acquaint himself with the meanings of all the terms in the list of members before proceeding farther.

The object of this chapter is to give a general description of the bridges with which this treatise deals and an explanation of how the different parts are connected. The floor system proper (i. e. the shims, ties, rails and guard rails with their connections) is described in the next chapter.

There are but four styles of bridge recommended viz. the plate girder for spans below sixty feet in length, the pony truss from sixty to seventy or eighty feet, the single intersection or Pratt truss from seventy or eighty to one hundred and eighty feet and the double intersection or Whipple truss from one hundred and eighty to three hundred feet.

The first style is so common in Japan that but little explanation is necessary, a reference to Plate VII rendering clear the design. The special feature of the girder there represented is the inclined stiffening angle which bears upon the bed plate near its inner edge. Its object is to prevent a bending of the flange at the edge of the bed plate. That some such device as this is necessary the author was once convinced by inspecting some steel girders on the Atchison, Topeka and Santa Fe R. R.: they had bent quite perceptibly at the bearings.

The transverse bracing shown on Plate VII is different from that of the present Japanese plate girder bridges. The latter is sufficiently stiff for spans up to say twenty-five feet in length, but the diagonal bracing shown on that plate will be found to give superior rigidity for rapidly passing loads on longer spans. The pony truss of which some details are shown on Plate IX, is almost the only style of bridge except the plate girder which is used in Japan. There is considerable difference, however, between those of this country and those here recommended. The former are of the Warren type having bottom chords built in a trough shape of plates and angles, and the diagonal struts of two very wide eye-bars trussed. There is no lateral system or even side bracing, and the passing loads produce bending in the lower chords. The style here recommended differs from the through bridge which

will be described presently, by only the absence of upper lateral bracing and the addition of side braces. Although the "General Specifications" of Chapter VI provide for spans having less than five panels, the author does not recommend their use, even if a small saving of iron be accomplished thereby. The objection to three and four panel trusses is the unsightly appearance caused by the flat slope of the batter braces.

The Pratt or single intersection quadrangular truss has vertical intermediate posts, inclined tension web members, parallel chords and inclined end posts. The diagonal ties extend over one panel only, and the hip verticals are tension members.

The Whipple or double intersection quadrangular truss differs from the preceding only in the diagonal ties which extend over two panels and are generally halved and attached by pins to the middle of the posts, so that the assumed length of the latter as struts may be reduced to one half their real length.

It is better and more economical to build deck bridges with inclined end posts than with vertical ones, the track being carried over the piers by iron bents resting thereon. It is better because with vertical end posts there can be no stress in the end panels of the bottom chords, an undesirable state of affairs when vibration and wind pressure are considered.

In this and in all other cases where the floor system is made continuous over the piers of consecutive spans, expansion of the floor system must be provided for; otherwise injurious thermal stresses would be induced.

To all the styles of bridge except the plate girder the following explanations will apply.

Top chords are composed of two channels having their webs vertical with a plate rivetted above and latticing or lacing below as shown on Pl. IX Figs. 1 and 7. Where two channels and a half inch plate do not give a sufficient sectional area to the chord, an I beam of the same depth as the channels may be inserted between the latter as shown on Pl. IX Fig. 2 or, if this does not give sufficient area, channels may be built of plates and angles as in Pl. IX, Fig. 3. What is said of top chords applies also to batter braces.

Posts are composed of two channels with latticing or lacing on each side, as in Pl. II Figs. 10 and 11. When sufficiently large rolled channels are not procurable it is better to built them of plates and angles rather than to use any combination of I beams and channels.

Lateral and portal struts are also composed of two channels laced or latticed as shown on Pl. I and in detail on Plates II and VIII.

Intermediate struts for single track bridges are composed of single I beams with their webs horizontal (Pl. VIII, Figs. 2 and 4), and those for double track bridges of two channels laced or latticed.

Bottom chords consist principally of eye bars, but, as will be seen farther on, they may have to resist a certain amount of compression under exceptional circumstances, so that in most cases of single track bridges there is a strut of two laced channels lying between the eye bars and symmetrically arranged in reference to the

plane of the truss. (see Plate IX, Fig. 11). In short spans this strut may be omitted by trussing the inner chord bars, as shown on Plate IX, Fig. 10.

Side braces may be made of angle or tee iron, but preferably of channel iron as shown on Pl. IX, Figs. 16 and 17.

Floor beams and most track stringers are built of plates and angles as shown on Plates III and IV, but, when the panel length is short, I beams can be advantageously employed for the track stringers.

Main diagonals are always eye bars, and counters are of square or round section with loop eyes at the ends.

Hip verticals may be of round, square or flat iron: it is customary to make them narrower and thicker than main diagonals or chord bars, perhaps because iron in such a shape is stronger than when rolled into thin bars, and much strength is needed in these members to resist shock. Counters are always adjustable, i. e. are provided with an arrangement by which they may be shortened or lengthened, but main diagonals, chord bars and hip verticals are of invariable length.

Lateral and vibration rods are nearly always round, but there is no particular objection to making them of square iron: they are always adjustable.

It is not considered good practice to vary the thickness of the top chord plate or to increase the number of thicknesses towards the middle of the span; for the proper place for the larger part of the material in a top chord is in the channels and not in the plate. Similarly in any channel the best place for the larger part of the material is in the flanges and not in the web, nevertheless both thick and thin channel webs are necessary for the same structure.

Channels in struts generally have their flanges turned outward, although, theoretically, as far as the strength of the strut is concerned, it is better to turn them the other way, for the moment of inertia of the section is greater; nevertheless the difficulty of rivetting in a confined space more than equalizes the advantage thus gained. But there are certain cases, for instance in bottom chord struts, in which it is better to turn the flanges inward notwithstanding the difficulty in rivetting.

It is not well to reduce to any extent the widths of the flanges at the ends of strut channels, as thereby the strut is weakened; but a slight trimming may be effected, if the channels be well re-inforced.

Main diagonals, as will be demonstrated in Chapter XIII, should have the proportion of width to depth about one to four, and the chord bars that of from one to four to one to seven, according to the number of them in the panel.

Top chords except for very short spans are made up of lengths equal to a panel length, so there is a joint in the channels and plate near every panel point. This joint is placed three or four inches from the panel point and towards the nearer end of the span, in order that the pin hole may not be bored through two abutting ends, as is unavoidable at the hip joint. The abutting ends of each pair of channels are connected by a splice plate or connecting plate on each side of the web, making four plates to an ordinary joint besides the cover plate which connects the abutting ends of the chord plates. This detail is represented on Pl. IX, Fig. 7.

The connection at the hip is made by means of two plates on the outside of the

chord and two on the inside of the batter brace, through all of which the pin passes. Those on the chord abut against plates rivetted to the outside of the batter brace channels; and those on the batter brace abut against plates rivetted to the inside of the chord channels, all abutting surfaces being planed to fit exactly, so that, when the pin is driven into place, the whole joint is as rigid as if it were rivetted, and the batter brace may be proportioned as if fixed at the hip. There is also a cover plate to protect the hip joint from the weather and to add slightly to the rigidity of the connection. This detail is illustrated on Pl. IX, Fig, 4.

The posts are attached to the upper chord pins by extension plates rivetted to the ends of their channels, which ends are planed so as to fit closely up to the flanges of the chord channels. At their lower ends the bottom chord pins pass through holes in the webs of the channels, the centres of which holes are located from six to twelve inches from the end of the post. The webs are reinforced by plates on the inside and outside, which plates are turned up horizontally so that they can be rivetted to the floor beam, in order to resist any tendency that there may be to twist the post when the lateral rods are in action. These details are given on Pl. IX, Figs. 7 and 12 and on Pl. II, Figs. 9 and 10.

The joints in the bottom chord struts occur near the panel points, but because these struts usually act in tension it is well to have as few joints as possible and to make them extra strong.

The channels can be obtained long enough to span two panels. They will have to be slightly bent at the middle because of the cambre; this can be readily done by heating them. The connection is made by a reinforcing plate on each side of the strut, the plates being several inches deeper than the channels which they splice. These plates are used at each panel point even where there is no joint, so as to reinforce the webs of the channels at the pin hole. This detail is illustrated on Pl. IX, Fig. 11, and on Pl. II, Fig. 10.

At the pedestal the batter brace channels bear evenly against the horizontal shoe plate and are attached thereto by means of a bent plate shaped to fit accurately inside the channels and against the shoe plate. The batter brace upper plate projects beyond the channels, and is turned up horizontally so as to bear evenly against the shoe plate, to which it is rivetted. When channel struts are employed in the bottom chords, they connect on the shoe pins, and, if the latter be not supported, produce great bending moments thereon. It is therefore necessary to provide an intermediate bearing for the shoe pin. This can be done very conveniently by using an I beam with the upper flange removed, the lower flange being rivetted to the connecting plate and shoe plate. This detail is shown on Pl. II Fig. 12 and on Pl. IX, Fig. 13. The shoe plate at the fixed end of the span, if it be large enough to distribute the pressure, rests directly on the masonry which should be smoothly and accurately dressed at the bearing. If not large enough, it must rest upon a bed plate of wrought iron to which it is to be firmly bolted, the bolt heads resting in cavities in the masonry as shown on Pl. IX, Fig. 14.

The bed plate, or if there be none, the shoe plate must be firmly anchored to the masonry by bolts either built into the masonry, in which case there are washer

plates at their lower ends, or driven into holes drilled therein. In the latter case the ends of the bolts are split, wedges inserted in the slits, the bolts driven down hard so as to spread the ends; then molten sulphur is poured into the holes.

Expansion is provided for at the other peelestal in short spans by a tongue on the under side of the shoe plate fitting into a groove in the upper side of the bed plate or by such an arrangement as that shown on Pl. II, Fig. 15. In other spans the shoe plate rests upon a nest of turned rollers, held in a light frame and resting on a planed roller plate, which has angle iron rivetted around its edges so as to form a shallow box: this box is arranged so as not to hold water. The bottom of the shoe plate is so planed down as to leave along its centre line parallel to the length of the bridge a projecting rib about two inches wide and from an eighth to a quarter of an inch deep, which fits loosely into notches turned on the rollers. A similar projecting rib on the upper face of the roller plate also fits tightly into the notches on the rollers, effectually preventing lateral motion of any magnitude. The reason why the projecting portion of the shoe plate does not fit tightly into the notches in the rollers is because, if it did so and if the roller plates were not laid more accurately than can generally be done in practice, the end lower lateral strut might prove to be too long to enter the space assigned to it, or not long enough to fill it completely. Vertical motion is prevented by turning over the top of the *outer* angle of the roller plate so as to almost touch the top outer edge of the shoe plate. These pedestal details are all illustrated on Pl. II Fig. 12, and on Pl. IX, Fig. 13. Shoes are sometimes made hinged so as to make certain of there always being an even bearing upon the rollers, but most of the best bridge designers do not recognize the necessity for this refinement of construction: its use would cause an increase in the section required for the batter brace.

In double intersection bridges the long diagonals are halved and connected by pins, passing through the middle of the posts, the channels of which are reinforced by plates at the pin holes to compensate for the metal cut away. These holes should be slotted in the direction of the main diagonals, in order that the extension of the latter may cause no deflection of the post at the middle, but still permit of figuring the post as of half length with both ends hinged. The extension of the counters is not so important therefore the pin holes need not be slotted in the direction of their length. Centre posts, which are crossed by counters only, nearly always have a superabundance of strength, so their centre pin holes require no slotting.

Filling plates are used for top chords where there are abutting channels with webs of unequal thickness, in floor beams and track stringers to fill the spaces between stiffeners and webs, at pedestals to fill between the outer chord bar heads and the webs of the batter brace channels, and at first panel points of through bridges below the chord heads so as to make the floor beams at these places on the same level as that of the others.

Lateral struts are connected to chord pins both in the upper and lower systems by jaws as shown on Pl. VIII Figs. 1, 2 and 4. Where bent eyes are employed for the lateral rods there are two nuts of different diameters used for attaching the strut to the chord pin. The larger serves to hold the jaw against the chord, and the smaller

to resist the pull of the lateral rods: of course the pin has to be shouldered down, and it is evident that it should be inserted from the outside of the bridge. Jaw plates are either single or double; the former where a special vertical pin is used for the lateral rod attachment, the latter where the rods are connected to the chord pins by bent eyes. These bent eyes are not used on rods exceeding one and three quarter inches in diameter. The function of the inner jaw plate is to reduce the pressure of the bent eyes upon the nut on the end of the chord pin, and to avoid the oblique action of the same. This oblique action is not so objectionable in the case of light rods pulling against well proportioned bolts and nuts, as in the case of some vibration rod attachments.

Portal braces are connected to the batter braces by large short bolts through the medium of single jaw plates as shown on Pl. VIII Figs. 3 and. 6. When there are vibration rods at the portal the portal strut channels are placed closely together with their *webs* parallel to the plane of the batter braces, in which case one bolt through each jaw is sufficient; but, when there are no vibration rods, the portal strut channels are spread far apart with their *flanges* parallel to the plane of the batter braces, in which case two bolts through each jaw will be required. Portal vibration rods are attached to the struts by pins as shown on Pl. VIII, Fig. 3.

End upper lateral rods, if small, may be attached to the hip pins by bent eyes, provided that the nuts for said pins be well proportioned; but, when they exceed one and three quarter inches in diameter, they are coupled by split eyes to special vertical pins passing through the flattened ends of the hip pins as shown on Pl. VIII Fig. 5.

Intermediate vibration rods are attached to upper portal and intermediate struts by bolts or pins as on Pl. VIII Figs. 2 or 4.

Intermediate struts for single track bridges are connected to the posts by two bent plates at each end as shown on Pl. VIII Figs 2 and 4; those for double track bridges by jaws.

Knee braces or brackets are used to connect upper lateral, portal or intermediate struts to posts or batter braces. Where no vertical sway bracing is used these knee braces are very important portions of the structure, and have to be proportioned to resist calculated stress, but where sway bracing is employed their use is principally ornamental. They can be made of either angle, channel or tee iron; preferably the former: they are not to be bolted but rivetted to the parts which they connect. Side braces are to be connected to the top chords and floor beams as shown on Pl. IX, Figs. 16 and 17.

There are two different floor systems, to be described presently: in the first of these the lower lateral rods pass through the wooden shims; and the lower lateral struts between the ends of the track stringers; but in the second both rods and struts pass through holes in the webs of the stringers. The first system is shown on Pl. III the second on Pl. IV.

Floor beams are hung from bottom chord pins by beam hangers, formed of square iron with enlarged ends, which is bent into the shape of the letter U. The ends of the hangers pass through holes in a plate, called a beam hanger plate, which acts as a sort of stirrup for the floor beam. Nuts on the ends of the hangers are

turned so as to press the floor beam against the bottom of the post (or the filling piece at the first panel point) and are prevented from getting loose by lock nuts. This attachment is illustrated on Pl. II, Fig. 10. Floor beams in deck bridges should rest upon the top chords, directly over the posts: their lower flanges should be rivetted to the chord plate, and brackets of angle or channel iron should be used to prevent injury from the racking effect of passing trains. This detail is shown on Pl. IX, Fig. 15. It is not legitimate to use such floor beams as upper lateral struts, although they undoubtedly aid the upper lateral system.

The two floor systems referred to a few lines back are for these two cases viz., where there are wooden shims resting on the track stringers and where there are not. The former is employed when the stress in the end lateral rod is not sufficient to necessitate the use of double rods, and the latter in all other cases, the reason being that four rods passing through a shim would cut it up too much, besides making it inconvenient to get the rods and shims into place.

In the first case the track stringers rest on shelves of angle iron supported by short stiffening angles, and are also attached by bent plates rivetted to the webs of both beams and stringers. To obtain greater stiffness, plates of the full width of the top flanges may be run through the lateral struts and rivetted to the flanges of adjacent stringers, but these are not absolutely necessary and may be omitted, if so desired. This style of floor system is shown on Pl. III.

In the second case the stringers pass over the floor beams, which are a few inches lower than in the other case, and are made continuous from end to end of span by means of splice plates on both webs and flanges. Heavy stiffeners are placed beneath the points of support of the stringers to avoid all tendency to buckle the web of the beam. Additional support and rigidity are given to the connection by brackets extending from the bottom of the beam to the bottom of the stringers. This method is illustrated on Pl. IV.

All built track stringers have stiffening frames lying in planes transverse to their length and spaced from seven to ten feet apart. None are required at the ends, when the stringers abut against the floor beams, but there is one near each end when the stringers rest thereon. Extra stiff frames are required at the ends of stringers which rest upon the piers or abutments. Bed plates with grooves or some similar arrangement are to be used for the stringers where they bear upon masonry, and are anchored to same in a manner similar to that described for the case of pedestal bed plates. Similar bed plates are used at one end of plate girder spans. Lock nuts are used on all adjustable members: any style that will act efficiently may be employed, but those illustrated on Pl. II Figs. 5, 6 and 10 are quite simple and effective.

Ornamental work can be placed at the brackets on the portals, at the intersection of the portal vibration rods, above the upper portal struts, and even on intermediate brackets. A small amount of ornamental work will go a long way in an iron bridge.

In floor beams, track stringers and plate girders it is better to concentrate as much of the sectional area of the flanges into the angles and to use plates sparingly,

in spite of an apparent economy in employing the latter; for the iron acts more efficiently when the stresses thereon are not transmitted to too many parts.

Angle stiffeners for these members are preferable to tees or channels: they should be made flush with the vertical legs of the flange angles by filling plates, instead of being bent around the flanges, and at the same place there should always be one on each side of the web.

In making turn buckles, a little expense can be saved by having only one adjusting-end: the other having a hole, through which passes one end of the rod, which is enlarged into a head. One advantage of this style is, that the turn buckle can never be lost from the rod. Such a turn buckle should always be used on portal vibration rods, for a reason that will be given in Chapter XXII.

CHAPTER V.

FLOOR SYSTEM PROPER, RE-RAILING & DITCHING APPARATUS.

The floor system proper, or the arrangement of ties and guard rails, here recommended is not the one in common use in America, but is a modification of that proposed by W. Howard White, Esq. C.E., in the *Transactions of the American Society of Civil Engineers*, Nov. '83, the changes being the substitution of angle iron for the wooden guard rails, the shortening of the ties, the diminution of the spaces between them, and the providing of a place of safety for anyone who may be upon the bridge when a train is passing. The usual American floor system is much heavier than the one proposed by Mr. White, and consequently is not only more costly in itself, but by increasing the dead load of the bridge necessitates the use of more iron for the trusses. For short spans this might be considered advantageous, in that it tends to lessen vibration, but for long spans it is decidedly the contrary. The employment of long ties and outside guard rails necessitates the use of two extra stringers to sustain a portion of the live load in case of derailment, because the guard rails being higher than the rails must be placed at such a distance outside of the latter, in order to permit of the passage of snow-ploughs, that the unaided ties would not be strong enough to uphold a derailed locomotive.

Because of the flanges on the wheels an inner guard rail of the same height as the rail is fully as efficient as an outer guard rail two inches higher, and has the advantage that it may be placed close to the rail, thus preventing excessive lateral movement of a derailed carriage or locomotive.

The floor system recommended by the author is illustrated on Plates III and IV. Most of the ties are of 7″ × 8″ × 6′ oak, laid on their flats, and spaced not more than twelve inches centre to centre, thus leaving an opening of no more than four inches, which will not cause excessive jolting of derailed wheels. Every sixth or seventh tie in single track bridges is twelve feet long, so as to support at each end a 8″ × 12″ pine plank, extending from end to end of bridge, in order to afford a place of refuge from passing trains. Each foot plank is to be spiked to each long tie by two 7″ cut spikes.

In double track bridges the long ties are to be twenty-two feet in length, extending clear across the bridge, and supporting a run of plank at each end and another at the middle. The latter serves merely as a stepping place to pass from one track to the other, the outer runs only being intended for places of safety.

When wooden shims are used, as in Plate III, the ties at the panel points are made of 7″ × 14″ × 6′ timber so as to span the opening left between the ends of the stringers for the passage of the lower lateral struts. The ties are dapped about an inch onto the shims, which are generally of 7″ × 8″ oak, and are connected thereto by drift bolts of three quarter inch square iron driven into three quarter inch round holes, bored obliquely through both ties and shims. The bolts are provided with square heads, so that they may be withdrawn by a claw bar, when the timber is to be replaced. The wooden shims are useful in affording an easy means of attaching the ties to the stringers, besides adding somewhat to the strength and stiffness of the latter,—enough in any case to compensate for the loss of strength caused by the holes for the attaching bolts. The latter should be staggered and spaced about two feet apart, but there should be no bolt placed nearer than two feet to the middle of the panel, where the bending moment on the stringer is at a maximum. These bolts are to be ¾″ in diameter, and the holes through which they pass $\frac{13}{16}$″ in iron and 1″ in wood.

The guard rails are of 5″ × 4″ × ½″ angle iron, each weighing 14½* per lineal foot, the five inch leg being vertical, and the four inch leg perforated for the passage of the ¾″ bolts which attach it to alternate ties.

The distance between the inner face of the head of the rail and the outer face of the guard rail is six inches.

To avoid splitting the wood a small hole should be bored whenever a track spike is to be driven.

When shims are not used, the ties are attached directly to the stringers by bolts.

By glancing over the bills of iron and lumber in Chapter XVIII and choosing those weights which belong to the floor system proper, the weight of iron per lineal foot of span in this portion of the bridge will be found to be 37 pounds and the weight of lumber per lineal foot 195 pounds. When shims are not used the weight of iron is almost unchanged, but the weight of lumber is reduced about 40 pounds per lineal foot·

Plate V illustrates a re-railing device to be placed at each end of a bridge. By its use any car or locomotive which is off the track, *at a distance not greater than half the gauge*, will be returned thereto before coming upon the bridge. This ingenious device is the design of R. McClure, C. E. Esq. use larger letter, Chief Engineer of the Chicago, Burlington and Quincy R. R. system, to whose kindness the author is indebted for the drawings from which Plate V was prepared. The author has been obliged to introduce several unimportant modifications to adapt it to the change of gauge and to the peculiar guard rail employed.

It consists essentially of an ordinary frog point placed midway between the rails at a short distance from the end of the bridge, from which point diverge two ordinary rails produced until their centre lines approach the centre lines of the track rails within about a foot. The ends of the former rails rest on an extra wide tie, and between each of them and the nearest track rail is a ⅜″ plate resting on two ties and having the end bent down. This plate joins onto a 4″ × 6″ angle iron with the four inch leg vertical, and cut and bent as shown on the drawing, the outer

vertical face being made continuous with the outer face of the rail head. The angle iron and a portion of the plate rest upon and are attached to a 5″ × 9″ oak timber laid on its flat, dapped and well spiked to the ties, and having the end bevelled off so as to make the plate and angle iron form an inclined plane. The other end is also bevelled off, but more suddenly. The vertical legs of the angle irons are so trimmed as to have their upper edges horizontal, and the other legs so that they will not approach too closely to the track rails. The outer faces of the vertical legs approach the inner faces of the track rail heads till the distance between them is two and a half inches, then run parallel thereto for a short distance and again diverge. The greatest elevation of the upper face of the six inch leg of the angle iron is three quarters of an inch less than that of the track rails.

On the outside of each track rail and close up to its lower flange is a ¼″ plate resting upon an oak timber 4½″ deep and of varying width: the width of the plate also varies. The timber is bevelled off at one end so as to form an inclined plane leading up to a level surface of the same height as that of the rails. The outer plate begins a few feet further from the bridge that does the inner one, and its rise is greater and more rapid, thus giving the derailed carriage or locomotive a cant towards the track, which with its momentum helps to throw it back on the rails.

The *modus operandi* is as follows: a derailed car, for instance, approaches the bridge, and the wheels which are between the rails strike either the frog point or one of the diverging rails. These direct the wheels unto the inclined planes up which they mount and upon which they are conducted by the vertical legs of the angle towards the track, approaching it so closely that the outer portion of their peripheries will rest upon the rails as soon as the angle iron begins to descend. The space between the outer plate and the head of the rail is so small that the flange of the wheel has no chance to enter it, so that the car cannot but regain its place upon the rails.

Mr. McClure has experimented upon this apparatus with perfect success, but has not yet adopted it on his roads; because it infringes upon a safety switch device already patented.

But as American patent laws do not extend as far as Japan, there seems to be no objection to the use of the apparatus on Japanese roads.

It is possible though not probable that a car may jump the track when on the bridge, to provide for which case the author has adapted the apparatus so as to re-rail such a car: this portion of the design may, however, be omitted without incurring any possibility of injury to the bridge.

There is still one case in which a derailed locomotive might injure a bridge even with this appliance in use; that is, when the wheels are more than half the width of the gauge out of line. There is nothing to be done in such a case except to ditch the train at the nearest comparatively safe place to the bridge. This can be done very easily by the apparatus shown on Plate VI, a design of the author's, which is probably also an infringement on one or more American patents. Its action is very simple: one of a pair of diverging angle irons, beginning at a distance from the track rails equal to half the gauge, catches the wheel flanges on one side

of the derailed vehicle and throws it clear of the track. The inclined planes on the inside of the track enable the other wheels to jump the rails.

Stiffening angles, with the vertical legs turned down so as to offer no obstruction to passing wheels, are used where there exists the greatest tendency to tear up the deflecting angles. They are to be well spiked to the ties.

Plates V and VI are not intended to be complete drawings in respect to detail, for the spikes, fish plates &c. are omitted. These details are, however, not essential to a thorough understanding of the devices illustrated.

From the ditching to the re-railing apparatus there should be a rail midway between the track rails to prevent any derailed vehicle, which passes the ditching apparatus, from diverging more than half the width of the gauge before reaching the frog point of the re-railing apparatus.

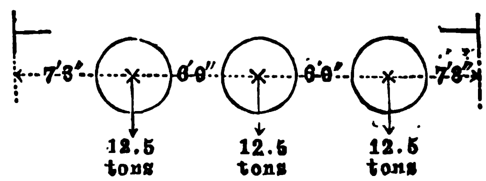
By the use of this combination perfect safety from derailment is insured to a bridge.

CHAPTER VI.

GENERAL SPECIFICATIONS.

Moving Load.

The moving load for each track is to consist of two engines, whose weights and distribution of same are given in the accompanying diagram



followed by a train of cars whose weights per lineal foot of track are to be taken from the following table

Span	Live Load per lineal foot.
Under 150'	1200 pounds.
From 150' to 200'	1150 „
„ 200' to 250'	1100 „
„ 250' to 300'	1050 „

To simplify calculation spans from twelve (12) to sixty (60) feet may be calculated for the uniformly distributed loads given in the following table

Span.	Unif. Load.	Span.	Unif. Load.
12' to 18'	4200 pounds.	33' and 34'	3400 pounds.
19' and 20'	4100 „	35' „ 36'	3300 „
21' „ 22'	4000 „	37' „ 38'	3200 „
23' „ 24'	3900 „	39', 40' and 41'	3100 „
25' „ 26'	3800 „	42', 43' „ 44'	3000 „
27' „ 28'	3700 „	45', 46' „ 47'	2900 „
29' „ 30'	3600 „	48', 49' „ 50'	2800 „
31' „ 32'	3500 „	51' to 60'	2700 „

The live load stresses in floor beams and track stringers and in all plate girders not exceeding twenty-five (25) feet in length are to be increased by twenty-five (25) per cent., in order to provide for shock.

For the same reason the stresses in plate girders of and exceeding twenty-five (25) feet in length are to be increased by the percentages given in the following table

Span.	Percentage.	Span.	Percentage.
25′ and 26′	24	37′ and 38′	18
27′ „ 28′	23	39′, 40′ and 41′	16
29′ „ 30′	22	42′, 43′ „ 44′	14
31′ „ 32′	21	45′, 46′ „ 47′	12
33′ „ 34′	20	48′, 49′ „ 50′	10
35′ „ 36′	19	51′ to 60′	8

Dead Load.

The dead load is to include the weight of all the iron and wood in the structure, excepting those portions resting directly on the abutments, and whose weights do not affect the stresses in the trusses.

Oak lumber is assumed to weigh four and a third ($4\frac{1}{3}$) and pine two and a half ($2\frac{1}{2}$) pounds per foot board measure. Should, in any bridge of or below two hundred (200) feet span, the calculated dead load differ more than seven (7) per cent., or in any bridge above two hundred (200) feet span more than four (4) per cent. from that assumed, the calculations of stresses &c. are to be made over with a new assumed dead load.

Wind Pressure.

The wind pressure is to be divided into two parts, one a moving load of two hundred and forty (240) pounds per lineal foot and the other a fixed load of thirty (80) pounds per square foot of exposed bridge surface. The latter is to be determined by adding together the area of the elevation of the floor system, figured by multiplying the vertical distance from the top of the rail to the bottom of the track stringer by the length of the span, and twice the area of the vertical projection of one truss.

Trusses of less than two hundred (200) feet span are also to be proportioned for a pressure of fifty (50) pounds per square foot when unloaded, those of from two hundred (200) to two hundred and fifty (250) feet for forty-five (45) pounds per square foot, and

those of two hundred and fifty (250) to three hundred (300) feet for forty (40) pounds per square foot. The area subjected to wind pressure is to be figured as just described; and the greater stress by either method of computation is to be used in determining the sectional area of the bracing.

In through bridges and pony trusses the wind pressure on the train is to be assumed as taken up by the lower lateral system, and in deck bridges by the upper lateral system. The wind pressure on the bridge itself is to be divided between the upper and lower systems according to the proportion of total area above and below the middle horizontal plane of the structure.

Stresses due to Curvature.

When a structure is upon a curve the stresses in chords and lateral systems and in the end transverse sway bracing of deck bridges due to the centrifugal force of the heaviest assumed train moving at a velocity of sixty (60) miles per hour are to be provided for. These stresses are to be considered of as great importance as any other stress in the members affected thereby.

Clear Roadway.

In all through bridges and pony trusses the clear distance between nearest members of trusses shall be at least twelve (12) feet six (6) inches for single track bridges and twenty-two (22) feet for double track bridges.

When such a structure is upon a curve, these minimum widths are to be increased by the product of the radius and the versed sine of one half the arc of the curve which is subtended by the span, plus seven (7) times the super-elevation of the outer rail above the centre line of the track.

Length of Span.

The length of span is to be understood as the distance between centre of end pins for trusses and between centres of bearing plates for all rolled beam and plate girder spans.

Limiting Lengths of Span for Different Clear Roadways.

The greatest lengths of span for the different clear roadways are to be taken from the following table

Clear Roadway.	Greatest Length of Span.
12.5′	75′
13.0′	120′
13.5′	150′
14.0′	180′
14.5′	210′
15.0′	240′
15.5′	270′
16.0′	300′

Clear Headway. The clear headway or vertical distance between the upper surface of the rails and the lowest part of the overhead bracing shall be at least twelve (12) feet six (6) inches.

Styles of Bridge for Different Spans. Spans below fifteen feet are to consist of rolled beams; spans between fifteen (15) and sixty (60) feet of rivetted plate girders; spans between sixty (60) and seventy (70) or eighty (80) feet of pony trusses or deck bridges, and spans above seventy (70) or eighty (80) feet of through or deck truss bridges.

Limiting Depths of Pony Trusses. The greatest allowable depth, measured from centre to centre of chords, for pony trusses is nine (9) feet.

Limiting Slope for Batter Braces of Pony Trusses. The least allowable slope for batter braces of pony trusses is to be two (2) horizontal to one (1) vertical.

Limiting Length of Span for Double Intersection Bridges. The least allowable lengths of span for double intersection trusses are one hundred and seventy (170) feet for single track bridges and one hundred and fifty (150) feet for double track bridges.

Side Braces. The least allowable batter for side braces in pony trusses is five (5) inches to the foot; and all side braces are to be made to resist both tension and compression. In no case is a side brace to have less strength than that of a 3″ × 3″ — 7* angle iron. All pony trusses are to be provided with side braces.

Limiting Sizes of Sections. No round rods less than one (1) inch in diameter nor square ones less than seven eigths ($\frac{7}{8}$) of an inch on a side are to be used: no bars less than five eigths ($\frac{5}{8}$) of an inch thick for diagonals, no large plates less than five sixteenths ($\frac{5}{16}$) of an inch thick, nor, except for filling plates, any iron less than a quarter of an inch in thickness anywhere in a bridge. The least allowable depths for channels are four (4) inches for lateral systems, five (5) inches for posts in pony trusses and six (6) inches for posts in trusses of through and deck bridges.

Limiting Widths of Plates. The unsupported width of any plate subjected to compression must never exceed thirty (30) times its thickness.

Limiting Sizes of Upper Lateral Rods. The least allowable sizes of upper lateral rods are those given in Table XIII.

Expansion. All spans shall be provided with some means of expanding and contracting longitudinally with a variation in temperature of one hundred and fifty (150) degrees Fahr. as must also the track stringers where they rest upon the piers or abutments.

Spans of over seventy-five (75) feet in length are to have at one end two nests of turned wrought iron friction rollers running between planed surfaces.

Anchorage. One end of every span must be firmly anchored to the masonry, and the other end must be so attached thereto as to prevent vertical and lateral and permit of longitudinal motion.

At each bearing there are to be four anchor bolts firmly attached to the masonry, the diameters being not less than seven eighths ($\frac{7}{8}$) of inch for plate girders, one (1) inch for pony trusses and one and a quarter ($1\frac{1}{4}$) inches for through and deck bridges.

Except in the case of swing bridges consecutive spans are not to be made continuous over the points of support. Continuous Spans.

The cambres for bridges of the different spans are to be taken from the following table. The cambres of the rails may be reduced to two thirds of these amounts by varying the depths of the daps in the ties. Cambre.

Span in feet.	Cambre in inches.
50— 70	1.0
70—100	1.5
100—130	2.0
130—160	2.5
160—190	3.0
190—220	3.5
220—260	4.0
260—300	4.5

In all deck bridges and in all through bridges, where the depth from centre to centre of chords is twenty-four (24) feet or over, vertical sway bracing is to be used. It is to be proportioned to resist the stresses produced by the wind pressure; and, in double track bridges, also those produced by the greatest inequality of panel loading. In deck bridges where the track is on a curve, the vertical sway bracing is to be proportioned to resist the stresses due to centrifugal force. Vertical Sway Bracing.

Portal and lateral struts subjected to bending must first be proportioned for direct stresses due to both wind pressure and the initial tensions on the rods meeting at the end of the strut, and then to their sections must be added sufficient area to resist the bending, the intensity of working bending stress being taken equal to six (6) tons. Portal and Lateral Struts Subjected to Bending.

The effect of wind pressure on posts and batter braces of double track bridges need not be considered; nor need it be considered in single track bridges, unless the section which it calls for exceed fifty Effect of Wind on Posts and Batter Braces.

(50) per cent of that needed to resist the live and dead load stresses; the intensity of working bending stress being taken as five (5) tons, and that for the transferred load stress as given in Table VIII. In case the area required to resist the wind stresses exceed fifty (50) per cent of that required to resist the live and dead load stresses, the total section is to be obtained by adding to the former area one half ($\frac{1}{2}$) the latter area.

Effect of Wind Pressure on Chord Stresses.

The effect of wind pressure on bottom chord tension in double track bridges need not be considered, but in single track through bridges the bottom chords are to be proportioned first for the sum of the live and dead load stresses with an intensity of five (5) tons, then for the sum of the live load, dead load transfered load and wind stresses with an intensity of seven and a half ($7\frac{1}{2}$) tons. If there be stresses due to centrifugal force the areas found as just described are to be increased by amounts sufficient to resist the last named stresses using an intensity of working stress of five (5) tons.

In case of single track deck bridges the upper chords are to be proportioned to resist the whole of the live and dead load stresses, or six tenths ($\frac{6}{10}$) of same plus the wind stresses minus the transfered load stresses whichever of these two quantities be the greater. If there be stresses due to centrifugal force, they are to be added to the greater of the two quantities already found. The intensity to be used is given in Table VIII. Wind pressure on top chords of through bridges need never be considered.

It is only the outer bottom chords of through and pony truss bridges and the inner top chords of deck bridges, in respect to the curve, that need to have their areas increased to resist the stresses produced by centrifugal force.

Initial Tensions.

To allow for the stresses caused in adjustable main members by the screwing up of the turn buckles or sleeve nuts, the stress in each such member is to be increased by the amount given in the following table

1" ⊙ — 1.00 tons.	$1\frac{3}{4}$" ⊙ — 2.50 tons.
$1\frac{1}{8}$" ⊙ — 1.25 „	$1\frac{7}{8}$" ⊙ — 2.75 „
$1\frac{1}{4}$" ⊙ — 1.50 „	2" ⊙ — 3.00 „
$1\frac{3}{8}$" ⊙ — 1.75 „	$2\frac{1}{8}$" ⊙ — 3.25 „
$1\frac{1}{2}$" ⊙ — 2.00 „	$2\frac{1}{4}$" ⊙ — 3.50 „
$1\frac{5}{8}$" ⊙ — 2.25 „	$2\frac{3}{8}$" ⊙ — 3.75 „
	$2\frac{1}{2}$" ⊙ — 4.00 „

Square or flat bars are to receive the allowance for round rods of equal sectional area.

Connection for Lateral Systems.

Lateral struts are always to be attached to the chord pins.

Lateral rods are also to be attached to the chord pins by bent eyes when the larger rod at the connection does not exceed one and three quarter (1¾) inches in diameter. In other cases they are to be connected to special vertical pins passing through jaws on the lateral struts. Floor beams are not to be used in any case as lateral struts.

Stresses in End Lower Lateral Struts.

In figuring the stress in a lower lateral strut at the roller end of a span, the stress caused by the total wind pressure on both upper and lower lateral systems is to be added to the transverse component of the initial tension in the end lateral rod or rods attached to the windward end of the strut, and from the sum is to be subtracted one fourth (¼) of the reduced pressure upon the windward shoe.

The calculations are to be made for the case of the bridge covered by the moving load and for that of the bridge empty, the greater of the resulting stresses being adopted.

Stiffened Bottom Chords.

In any panel of a bridge where the compression on the bottom chord due to the wind pressure when the bridge is empty exceeds the tension there due to the reduced dead load alone, the chord in that panel must be made to resist both the tension and the excess of compression.

Where two channels are employed for the lower chord, the effective areas of the webs alone are to be counted upon to resist tension.

Top Chord and Batter Brace Sections.

The top chord and batter brace sections, whenever large enough channels are procurable, are to consist of two channels with a plate above and latticing or lacing below. The top plates must be thin and of the same section throughout, the increase in sectional area being obtained by thickening the webs of the channels. For heavier chords, an I beam may be inserted between the channels, or the chord may be built of plates and angles in channel form, care being taken to remove the material as far as practicable from the neutral surfaces.

Post Sections.

Posts are to be made of two channels latticed or laced, when sufficiently large channels are procurable. Otherwise they are to be built of plates and angles in channel form, the same restriction in regard to a proper disposition of the material as in the case of chord sections being observed.

Upper Lateral, Lower Lateral and Portal Strut Sections.

Upper lateral, lower lateral and portal struts are to be composed of two channels laced or latticed, and are to be connected by properly proportioned jaws.

Intensities of Working Tensile Stress.

The intensities of working stress for iron in tension are to be taken from the following table.

Member.	Intensity.
Plain Chord Bars	5.00 tons.
Trussed Chord Bars	4.00 „
Adjustable Diagonals	4.00 „
End Main Diagonals	5.00 „
Other Main Diagonals between	4.00 & 5.00
Plain Hip Verticals.	4.00 „
Trussed Hip Verticals	3.00 „
Flanges of Rolled Beams	5.00 „
Flanges of Built Beams (net section)	4.00 „
Lateral and Vibration Rods	7.50 „
Beam Hangers	3.00 „

The intensities for intermediate main diagonals are to be interpolated according to position.

Intensities of Working Compressive Stress.

For struts composed of two channels with plates or latticing or lacing, or of two channels with an intermediate I beam, or struts built of plates and angles in a shape resembling that of the channel strut, the following formulae are to be used in finding intensities of working compressive stress.

1st. For lateral and intermediate struts.

$$p = \frac{\dfrac{f}{1 + \dfrac{H^2}{C}}}{8 + \dfrac{H}{80}}$$

and 2nd, for all other struts.

$$p = \frac{\dfrac{f}{1 + \dfrac{H^2}{C}}}{4 + \dfrac{H}{20}}$$

where p is the intensity of working compressive stress, H the length of strut divided by *least* diameter of same,

$$f = \begin{cases} 19.25 \text{ for two fixed ends} \\ 19.25 \text{ for one fixed end and one hinged end} \\ 18.90 \text{ for two hinged ends} \end{cases}$$

$$\text{and } C = \begin{cases} 5{,}820 \text{ for two fixed ends} \\ 3{,}000 \text{ for one fixed end and one hinged end} \\ 1{,}900 \text{ for two hinged ends} \end{cases}$$

The working loads for I beam struts are to be taken from Table X.

For the flanges of rolled beams the intensity is to be taken equal to five (5) tons, and for those of built beams four (4) tons on the gross section.

Intensities of Working Bending Stress.

The intensity of working bending stress for pins and rivets belonging wholly to the lateral systems or sway bracing is to be taken equal to eleven and a quarter ($11\frac{1}{4}$) tons, and for all other pins and rivets seven and a half ($7\frac{1}{2}$) tons.

Where steel pins of good quality are employed, the intensity may be assumed at twelve (12) tons for trusses and eighteen (18) tons for lateral systems.

Intensities of Working Bearing Stress.

The intensity of working bearing stress for pins and rivets belonging wholly to the lateral systems or sway bracing is to be nine (9) tons, and for all other pins and rivets six (6) tons.

Hip Verticals in Pony Trusses.

Hip verticals in pony trusses, having less than five (5) panels, are to be stiffened so as to resist compression due to initial tension on counters. If the section employed consist of two channels, the net section of the webs alone is to be relied upon to resist tension, and if trussed bars be employed the intensity of working tensile stress on the net section is to be reduced to three (3) tons.

Upset Rods.

Middle panel diagonals, counters, lateral rods, vibration rods and all other adjustable rods are to have their ends enlarged for the screw threads according to the dimensions given in Chapter XIX, and are to be provided with check nuts.

Threads.

All threads except those on the ends of pins must be of a uniform standard.

Trussing.

Trussing is to be employed only for stiffened bottom chords and hip verticals. The least allowable section for trussing bars is a quarter ($\frac{1}{4}$) of an inch by three (3) inches.

Least Dimensions for Chord and Batter Brace Plates.

The least dimensions for the upper plates in top chords and batter braces are to be taken from the following table.

With channels greater than seven (7) inches in depth, should the width of plate employed exceed that given in the table by from forty (40) to sixty (60) per cent., the thickness must be increased by one sixteenth ($\frac{1}{16}$) of an inch: if it exceed by from sixty (60) to eighty (80) per cent., the thickness must be increased by one eighth ($\frac{1}{8}$) of an inch.

Depth of Chord.	Least Thickness.	Least Width.
6″	$\frac{1}{4}$″	8″
7″	$\frac{1}{4}$″	9″
8″	$\frac{1}{4}$″	10″
9″	$\frac{5}{16}$″	$11\frac{1}{2}$″
10″	$\frac{5}{16}$″	$12\frac{1}{2}$″
12″	$\frac{3}{8}$″	15″
15″	$\frac{3}{8}$″	$18\frac{1}{2}$″
18″	$\frac{3}{8}$″	$22\frac{1}{2}$″
20″	$\frac{7}{16}$″	$24\frac{1}{2}$″
22″	$\frac{7}{16}$″	27″
24″	$\frac{1}{2}$″	29″

Stay Plates. Sizes of stay plates are to be taken from Table XXII or XXIII. Stay plates on latticed or laced compression members are to be placed as near the pin holes or ends of strut as possible.

Latticing or lacing must never be used without stay plates at the ends.

Lattice and Lacing Bars. Sizes of lattice and lacing bars are to be taken from Tables XX and XXI. Lattice bars shall make with each other, as nearly as circumstances will permit, angles of ninety (90) degrees, and lacing bars angles of sixty (60) degrees.

The ends of lattice bars and single rivetted lacing bars are to be semi-circular, the centre being taken a little outside of the outer edge of the rivet hole.

Diameters of Rivets for Different Channels. For attaching plates and lattice or lacing bars to the flanges of channels the least diameters of the rivets to be used are to be taken from the following table; and the greatest diameters must not exceed those there given by more than one eighth ($\frac{1}{8}$) of an inch.

Depth of Channels.	4″	5″	6″	7″	8″	9″	10″	12″	15″
Dia. of Rivets.	$\frac{1}{2}$″	$\frac{1}{2}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″

Built Channels. Channels built of plates and angles are not to be less than eighteen (18) inches in depth, nor have webs less than one half ($\frac{1}{2}$) inch thick, nor flanges less than three (3) inch by four (4) inch angles weighing thirteen (13) pounds per lineal foot, the shorter legs to be rivetted to the web. For all built channels there are to be employed, instead of the ordinary latticing or lacing bars, pieces of angle iron having dimensions not less than two (2) inches by three and a quarter ($3\frac{1}{4}$) inches by a quarter ($\frac{1}{4}$) of an inch or weighing four and two tenths ($4\frac{2}{10}$) pounds per lineal foot, used as lacing bars and attached to the flanges of the built channels by two-staggered rivets at each end.

Splice Plates. The length of a splice plate is to be determined by the number of rivets necessary to transfer the stress from one main member to the other: the sum of the working bearing resistances of all the rivets on either side of the joint must not be less than the stress in the main member upon that side. The rivets must also be figured for bending.

When practicable, a splice plate must be placed on each side of every member where a joint occurs.

The transmission of compressive stresses shall be considered as entirely through the medium of the rivets and connection plates, and these must be proportioned accordingly; so that the area of the

section of the two splice plates connecting two channel bars must be at least equal to that of the larger channel.

Reinforcing Plates.

Simple reinforcing plates or plates rivetted to webs at pin holes in order to compensate for strength lost there, or to provide additional bearing for the pins must have as many rivets to attach them to the webs as will give bearing and bending resistances for same equivalent to at least the greatest stresses that can come upon the reinforcing plates.

Cover Plates.

Cover plates for top chords or batter braces are to have the same section as the chord or batter brace plate, the joints in which they cover, and enough rivets on each side of every joint to take up the greatest stress that could ever come upon the cover plate.

Extension Plates.

Extension plates on the end of a strut, for the purpose of hinging the latter, are to have from the pin holes to the nearest edges of the stay plates at least twice the sectional area of the strut, and the thickness must be sufficient to give proper bearing upon the pin. The length of the extension plates is to be such as to allow of the use of enough rivets to provide the proper bearing and bending resistances.

Shoe Plates, Roller Plates, and Bed Plates.

The thicknesses of shoe plates and roller plates for batter brace channels of the various depths are to be taken from the following table

Depth of Channel.	Less than 9″	9″ and 10″	12″	15″	Built Channels.
Thickness of Plate.	$\frac{3}{4}$″	$\frac{7}{8}$″	1″	$1\frac{1}{8}$″	$1\frac{1}{4}$″

Bed plates at pedestals must be of such dimensions that the greatest pressure on the masonry shall not exceed two hundred (200) pounds per square inch, and the thickness is to be the same as that of the shoe plate resting thereon.

Beam Hanger Plates.

Beam hanger plates are never to be made less than one (1) inch thick, and their areas are to be such that the hanger nuts will always have a full bearing thereon.

Rivetting.

Rivets in top chord and batter brace plates shall be spaced four (4) diameters apart for a distance on each side of every joint equal to one and a half ($1\frac{1}{2}$) times the width of the plate, and no more than five (5) inches elsewhere.

The distance between the edge of any plate and the centre of a rivet hole must never be less than one and a half ($1\frac{1}{2}$) times the diameter of the rivet, preferably twice the diameter.

The diameter of the hole shall never exceed that of the rivet by more one sixteenth ($\frac{1}{16}$) of an inch.

When two or more thicknesses of plate are rivetted together in compression members, the outer row of rivets shall not be more than three (3) diameters from the side edge of the plate.

Rivet holes must never be spaced more than two and a half ($2\frac{1}{2}$) seldom less than three (3) diameters, nor more than six (6) inches from centre to centre.

All the rivet holes of the respective parts of any structure must be made to exactly coincide either by drilling the holes full size through the connecting portions after being put together, or by sub-punching the pieces separately and afterwards reaming the combined rivet holes to proper size. In all cases the burrs must be removed by slightly countersinking the edges of the holes.

All rivets in splice or tension joints are to be systematically arranged, so that each half of a tension member or splice plate will have the same uncut area on each side of its centre line.

No rivet is to have a less diameter than the thickness of the thickest plate through which it passes, except in the case of thick shoe plates where the holes must be drilled.

The least allowable diameter for a rivet is half ($\frac{1}{2}$) an inch.

Rivets must not be used in direct tension.

Floor Beams, Built Track Stringers and Plate Girders.

Floor beams, built track stringers and plate girders must be well stiffened at the points of support and at intervals about equal to the depth of the beam.

The stiffeners are to be of angle iron and are to be placed in pairs on opposite sides of the web. They must extend from the upper leg of the upper flange to the lower leg of the lower flange of the beam, being made flush with the other legs of the flanges by filling plates.

Rivets in Flanges of Floor Beams &c.

In spacing the rivets in the flanges of floor beams, track stringers or plate girders, the beam is to be divided into equal portions of about two (2) feet in length, the stresses in the flanges are to be found at the points of division, and there must be enough rivets between any consecutive points of division to take up a stress equal to the resultant of the difference of the stresses at the points and the greatest vertical load that can come on this length, providing that the rivets be not spaced more closely than two and a half ($2\frac{1}{2}$) diameters.

Functions of Webs and Flanges.

In calculating the dimensions of a floor beam, iron track stringer or plate girder, the web must have sufficient area to take up the greatest shear, and the flanges alone must be assumed to resist the bending. The intensity of working shearing stress is to be two (2) tons.

Limiting Thickness of Webs.

No web for a floor beam, track stringer or plate girder is to have a less thickness than three eighths ($\frac{3}{8}$) of an inch.

Webs and flanges of floor beams, track stringers and plate girders must be well spliced at all joints by a plate on each side of the part spliced.

Splices in Floor Beams &c.

There is to be line of track stringers directly under each rail.

Track Stringers.

They are to be very firmly attached to the floor beams, and at the ends of the span are to rest upon and slide in grooved bed plates, or some similar detail.

Track stringers over fifteen feet in length must be braced by an angle iron frame or frames placed transversely to the span and firmly rivetted to the webs of the parallel stringers. Where the stringers rest upon the tops of the floor beams, there must be a bracing frame near each end of each pair of stringers. A similar bracing with the frames not more than seven (7) feet apart will suffice for plate girder spans not exceeding twenty-five (25) feet, but beyond that length there must be provided diagonal bracing of angle iron both above and below, the braces making with each other angles of about sixty (60) degrees. There must be also a stout bracing frame at each end of the span.

Bracing for Track Stringers and Plate Girders.

Plate girders and built stringers are not to have a depth less than one tenth ($\frac{1}{10}$) of the span wherever the headway beneath the track will permit the use of such a depth.

Limiting Depths of Girders & Stringers.

The flange-plates of all girders must be limited in width so as not to extend beyond the outer lines of rivets connecting them to the angles more than five inches or more than eight times the thickness of the first plate. Where two or more plates are used on the flanges, they shall either be of equal thickness or shall decrease in thickness outward from the angles.

Flange Plates of Girders.*

In welded heads the length of metal behind the pin must be at least equal to the diameter of the pin: while in hammered heads, the amount is to be the same as that above or below the pin.

Eyes.

The least amount of metal in the heads across the pins is given in the following table.

Width of Bar.	Diameter of Pin.	Metal in Head.	Across Pin.
		Welded.	Hammered.
1	0.80	1.50	1.50
1	1.04	1.50	1.50
1	1.12	1.50	1.53
1	1.20	1.50	1.56
1	1.28	1.50	1.60
1	1.36	1.55	1.72
1	1.43	1.60	1.76
1	1.50	1.67	1.85
1	1.64	1.67	1.95
1	1.77	1.70	2.05
1	1.90	1.76	2.21

* From the specifications of Theodore Cooper Esq., C.E.

In loop eyes the distance of the inner point of the loop from the centre of the pin must be not less than three times the diameter of the pin.

The loop must fit closely to the pin throughout its semi-circumference.

Pin holes in eye bars shall be bored to an exact size and distance, and to a true perpendicular to the line of stress; no error in the length of bar exceeding one sixty fourth ($\frac{1}{64}$) of an inch will be allowed, nor any variation of more than one sixty-fourth ($\frac{1}{64}$) of an inch between the centre of the eye and the centre line of the bar.

Pins.

Pins are to be proportioned to resist the bending produced in them by the bars or struts which they connect. Steel pins are also to be proportioned for shear.

No pin is to have a diameter less than eight tenths ($\frac{8}{10}$) of the depth of the deepest bar coupled thereon, nor shall it vary from that of the eyes of the bars coupled thereto by more than one fiftieth ($\frac{1}{50}$) of an inch.

The least allowable diameters for pins are two (2) inches for those belonging wholly to the lateral systems or sway bracing, and two and a half ($2\frac{1}{2}$) inches for all other pins.

Pin Bearing.

All pin holes shall be reinforced by additional material, so as not to exceed the permissible pressure on the bearings.

Where a pin bears against a reinforced channel bar, the web of the latter is not to be assumed to take up any bearing stress, unless the reinforcing plate or plates be rivetted to it before the pin hole be bored.

Chord Packing &c.

The lower chords are to be packed as closely as possible and in such a manner as to produce the least bending moments upon the pins. The various members attached to any pin must be packed as closely as possible, and all interior vacant spaces must be filled with wrought iron fillers.

Expansion rollers.

Expansion rollers are to be proportioned by the formula $p = 0.25\sqrt{d}$, where p is the working load in tons per lineal inch of roller, and d is the diameter of the roller in inches.

The least allowable diameter for rollers is two (2) inches.

The spaces between rollers must never exceed one half ($\frac{1}{2}$) of their diameter.

Turn-buckles and Sleeve-nuts.

All turn-buckles and sleeve-nuts must be made so strong that they would be able to withstand without rupture the ultimate pull of the rods which they connect. U nuts are not to be used in any part of a bridge.

Nuts.

The dimensions of all square and hexagonal nuts for the various diameters of rods are to be taken from Chapter XIX.

Those for the ends of pins, when not subjected to calculable

stress may be made of any convenient size and as thin as half ($\frac{1}{2}$) an inch. All nuts must have a uniform bearing.

Cast or wought iron washers must be used under the heads and nuts of all bolts in timber. Their bearing must be uniform. Washers.

Whenever possible four (4) beam hangers must be used per floor beam. The screw ends are to be upset and provided with check nuts. Beam Hangers.

In designing jaws special care must be taken to make them and their connections quite as strong as the body of the strut of which they form a part. Jaws.

Two brackets or knees must be used to connect each over-head strut to the posts or batter braces. Brackets.

They should be made of straight tee, angle or channel iron: if made curved, no dependence is to be placed on them for either strength or stiffness.

When there is no vertical sway bracing, each intermediate bracket must be proportioned to resist the compression induced in it by the wind pressure concentrated at the windward and leeward points of that panel of the top chord to which the bracket belongs; and each portal bracket must be proportioned to resist the compression induced in it by one half of the total pressure concentrated at the panel points of the top chords.

The flanges at the ends of channel bars must not be cut away, if it be practicable to avoid doing so; if not, there must be sufficient reinforcing used to make the strut as strong as it would have been with the flanges uncut. Cutting off the Flanges of Channels.

The arrangement of the parts of the floor system proper, viz. the ties, shims and guard rails with their connections must be as described in Chapter V. Ties must bear evenly upon their supports without the aid of filling pieces. Ties, Shims and Guard Rails.

Finally as regards the proportioning of any structure, if cases should occur, which are not covered by the preceding specifications, the following rule must in all such cases be adhered to: "Details must always be proportioned so as to resist every direct and indirect stress, that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress." Details not previously mentioned.

At each approach of every bridge there must be provided some arrangement to either return to the track or ditch derailed cars or locomotives. Re-railing and Ditching Apparatus.

No cast iron is to be used anywhere, except as washers for timber bolts. Cast Iron.

Field rivetting must be done with the button sett: the heads of the rivets must be hemispherical, and no rough edges must be left. Field Rivetting.

The amount of field rivetting must be reduced to a minimum.

Manufacturing Limitations.*

In punching plate or other iron, the diameter of the die shall in no case exceed the diameter of the punch by more than $\frac{1}{16}$ of an inch, and all holes must be clean cuts without torn or ragged edges.

All rivet-holes must be so accurately spaced and punched that when the several parts forming one member are assembled together, a rivet $\frac{1}{16}$ inch less in diameter than the hole can be entered, hot, into any hole, without reaming or straining the iron by " drifts."

The rivets when driven must completely fill the holes.

The rivet-heads must be hemispherical and of a uniform size for the same sized rivets throughout the work. They must be full and neatly made, and be concentric to the rivet-hole, and thoroughly pinch the connected pieces together.

Wherever possible, all rivets must be machine driven.

The several pieces forming one built member must fit closely together, and when riveted shall be free from twists, bends, or open joints.

All joints in riveted work, whether in tension or compression members, must be fully spliced, as no reliance will be placed upon abutting joints. The abutting ends, however, must be dressed straight and true so that there shall be no open joints.

Workmanship.*

All workmanship shall be first-class in every particular.

Abutting joints in truss bridges shall be in exact contact throughout.

Bars which are to be placed side by side in the structure, shall be bored at the same temperature and of such equal length that upon being piled on each other the pins shall pass through the holes at both ends without driving.

Whenever necessary for the protection of the thread, provision shall be made for the use of pilot nuts in erection.

Painting.*

All iron work before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and be given one good coating of pure boiled linseed oil, well worked into all joints and open spaces.

In riveted work the surfaces coming in contact shall each be painted before being riveted together. Bottoms of bed-plates, bearing-plates, and any parts which are not accessible for painting after erection shall have two coats of paint; the paint shall be a good quality of iron ore paint subject to approval of Chief Engineer.

After the structure is erected, the iron work shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure linseed oil, of such color as may be directed, the tension members being, however, generally of lighter color than the compression members.

* From the specifications of Theodore Cooper Esq., C.E.

Pins, bored pin-holes, and turned friction rollers shall be coated with white lead and tallow before being shipped from the shop.

All wrought iron must be tough, fibrous, and uniform in character. It shall have a limit of elasticity of not less than 26,000 pounds per square inch. Quality of Material.*

Finished bars must be thoroughly welded during the rolling, and be free from injurious seams, blisters, buckles, cinder spots, or imperfect edges.

For all tension members double rolled bars must be used. They shall stand the following tests :

Full sized pieces of flat, round, or square iron, not over 4½ inches in sectional area, shall have an ultimate strength of 50,000 pounds per square inch, and stretch 12½ per cent. in their whole length.

Bars of a larger sectional area than 4½ square inches, when tested in the usual way, will be allowed a reduction of 1,000 pounds per square inch for each additional square inch of section, down to a minimum of 46,000 pounds per square inch.

When tested in specimens of uniform sectional area of at least ½ square inch for a distance of 10 inches, taken from tension members which have been rolled to a section not more than 4½ square inches, the iron shall show an ultimate strength of 52,000 pounds per square inch, and stretch 18 per cent. in a distance of 8 inches.

Specimens taken from bars of a larger cross section than 4½ inches will be allowed a reduction of 500 pounds for each additional square inch of section, down to a minimum of 50,000 pounds.

The same sized specimens taken from *Angle* and other shaped iron shall have an ultimate strength of 50,000 pounds per square inch, and elongate 15 per cent. in 8 inches.

The same sized specimens taken from *Plates* less than 24 inches in width shall have an ultimate strength of 48,000 pounds, and elongate 15 per cent. in 8 inches.

The same sized specimens taken from plates exceeding 24 inches in width shall have an ultimate strength of 46,000 pounds, and elongate 10 per cent.

All iron for tension members must bend cold, for about 90 degrees, to a curve whose diameter is not over twice the thickness of the piece, without cracking. At least one sample in three must bend 180 degrees to this curve without cracking. When nicked on one side, and bent by a blow from a sledge, the fracture must be nearly all fibrous, showing but few crystalline specks.

Specimens from *angle*, *plate* and *shaped* iron must stand bend-

* From the specifications of Theodore Cooper, Esq., C.E.

ing cold through 90 degrees, and to a curve whose diameter is not over three times its thickness, without cracking.

When nicked or bent, its fracture must be mostly fibrous.

Rivets and pins shall be made from the best double-refined iron.

The cast iron must be of the best quality of soft gray iron.

All timber is to be of the best quality, free from wind shakes, large knots, decayed wood, sap: or any other defect that would impair its strength or durability.

Test of Structure.

Before the final acceptance of any bridge and after it has been in constant use for one day, it may be tested by passing over it the specified train or trains or their equivalent at a speed not exceeding thirty (30) miles per hour, and bringing them to a stop at any point by means of the air brakes or any brakes; or by resting the maximum train load upon the structure for twelve (12) hours. After such tests the structure must return to its original position without showing any permanent change in any of its parts.

CHAPTER VII.

LIVE AND DEAD LOADS, WIND PRESSURE.

By making inquiries of a number of railway officials the author has ascertained that the heaviest engine used in Japan weighs about thirty-three English tons. It is a tank engine about twenty-eight feet long and rests upon three pairs of wheels, which support the weight nearly equally, and whose axles are very nearly the same distance apart.

It is the opinion of all the officials consulted that this is about as heavy an engine as will ever be put upon the present Japanese roads, or upon any future roads built according to the same general design.

On this account the author has assumed the engine specified in the beginning of the last chapter as a standard for proportioning bridges. Its weight is thirty seven and a half American tons or about thirty three and a half English tons, so it is a trifle heavier than any engine in use in Japan. Such an engine carries its own coal and water, so is not followed by a tender. The engines with tenders are so much lighter than the tank engines that the combined weights of the former exceed that of the latter by only two or three tons, and are distributed over forty-two feet instead of twenty-eight, so that it is unnecessary to consider the effect upon bridges of the combined engine and tender loads.

It is unusual in this country, although quite common in America to couple two engines at the head of a train, nevertheless such an arrangement does occur here occasionally, so it has been adopted in this treatise; because bridges should be proportioned for the greatest loads that can ever come upon them.

In regard to car loads the information obtained shows that the freight cars when loaded bring the greatest loads upon the track, and that a loaded car weighs no more than eight tons twelve hundred weight or 19,264 pounds. The length of car from buffer to buffer is eighteen feet, making the greatest load per lineal foot 1070 pounds.

Now as cars are sometimes overloaded, although it is very bad for the springs and against the rules of the railroad, the author has thought it best to assume a car load of 1200 pounds per lineal foot for spans under one hundred and fifty feet in length, and to reduce it gradually to 1050 pounds for spans of three hundred feet.

The reason why such a reduction is permissible is because the chance of a long span ever being covered from end to end by the maximum load is very small, while

for short spans it is not at all improbable. Another reason is that as the length of span increases the ratio of dead load to total load increases and consequently the injurious effect of impact relatively decreases. This reduction, of course, does not affect the stresses on the floor system and hip verticals, which are proportioned for the assumed engine loads.

The table of equivalent uniformly distributed live loads given in the last chapter will be found to save much labour: although not perfectly accurate it is correct enough for all practical purposes. The table which follows it gives the percentages by which these equivalent loads are to be increased to resist the shock of rapidly passing engines. The percentages agree fairly well with the results of experiments upon the vibrations of bridges under passing loads. The two tables can be combined into the following, which is the one to be used in calculating stresses.

Span.	Unif. load.	Span.	Unif. load.
12′ to 18′	5250 pounds.	33′ and 34′	4080 pounds.
19′ and 20′	5125 "	35′ and 36′	3927 "
21′ and 22′	5000 "	37′ and 38′	3776 "
23′ and 24′	4875 "	39′, 40′ and 41′	3596 "
25′ and 26′	4712 "	42′, 43′ and 44′	3420 "
27′ anp 28′	4551 "	45′, 46′ and 47′	3248 "
29′ and 30′	4392 "	48′, 49′ and 50′	3080 "
31′ and 32′	4235 "	51′ to 60′	2916 "

The engine excess for one truss of a single track bridge is found by subtracting from the total weight of engine the product of the length of engine by the assumed car load per lineal foot and dividing the remainder by two; making the various engine excesses to be used as given in the following table.

Car Load.	1200* per ft.	1150* per ft.	1100* per ft.	1050* per ft.
Engine Excess.	10.35 tons.	10.7 tons	11.05 tons.	11.4 tons.

For double track bridges the engine excesses are, of course, twice as great as the above.

When the panel length is eighteen feet or over the engine excesses of the two coupled locomotives are assumed to be concentrated upon consecutive panel points, but for shorter panels there is supposed to lie between them a panel point without engine excess. Of course this division does not represent the actual distribution of the loading, nevertheless it is convenient and gives a small error on the side of safety. In double intersection trusses greater web stresses are found by assuming a car to be placed between the two engines, thus bringing both excesses upon the same system of posts and diagonals, so this arrangement was adopted when calculating the *web* stresses for Table IV.

Accurate values for the dead loads of single track bridges are given in Table I. The column for dead load is obtained by adding together the weights of iron per

lineal foot in the trusses, floor system and lateral systems, the weight of lumber per lineal foot and that of the rails, and subtracting from the sum 92 pounds to allow for the weights of those portions which rest upon the masonry, and which do not affect the stresses in the trusses. This allowance has been checked for several spans of various lengths, and has been found to be almost constant for all lengths of span.

By doubling the dead loads of Table I can be found approximate values for the dead loads of double track bridges. Although this method would be a rather rough approximation in calculating the weight of iron for a double track bridge from that of a single track bridge, still it will be found so accurate, as far as dead load is concerned, as not to necessitate a re-calculation of stresses. This is as one might anticipate; for the truss weights are almost directly proportional to the total loads, except when the change is so slight as to cause no alteration in the depth of the top chord and batter brace channels; the weights of the lateral systems, though not twice as great, are considerably greater in double than in single track bridges; for, although the total areas opposed to wind pressure are only slightly greater in the former than in the latter, yet all the members are longer, and those subjected to compression are consequently of larger sectional area; and the weights of the floor system are more than doubled; for those of all the members except the floor beams are doubled, while those of the latter are increased both by reason of their length and by the doubled loads thereon. The author has tested an actual case, and has found that both the weight of iron and the dead load are almost exactly twice as great in a double as in a single track bridge of the same span and live load per track.

From actual measurements of a number of cars the average area per lineal foot of train exposed to wind pressure has been found to be about eight square feet. The assumed wind pressure of thirty pounds per square foot on the train agrees with that of the best American practice: moreover it is as high as economical reasons will allow, for it would probably overturn any ordinary train, as the following calculations will show.

Let P = pressure per lineal foot of train
h = height of centre of pressure above rails
d = distance between centre lines of rails
and W = weight of car per foot which will just resist the over turning moment

then $Ph = \frac{1}{2} Wd$ and $W = \frac{2Ph}{d}$

The value of h is about seven feet, that of d exactly 8.7 feet and that of p two hundred and forty pounds.

Consequently $W = \frac{2 \times 240 \times 7}{8.7} = 908$ pounds.

As the greatest allowable car load per foot is only 1070 pounds, it is highly improbable that the average load would reach nine hundred pounds, so that a pressure of thirty pounds per square foot in upsetting a train on a bridge would destroy

the structure in any case, for bridges are not proportioned to resist the blows of derailed trains.

It is true that higher pressures than thirty pounds are sometimes recorded, but they extend over very limited areas: on this account the empty bridges are proportioned to resist from fifty to forty pounds per square foot according to the length of span. C. Shaler Smith Esq. C. E., one of the highest American authorities upon bridge building, proportions all his bridges under two hundred feet span to resist a pressure of fifty pounds per square foot, and considers that thirty pounds upon the loaded bridge will be large enough for all greater spans.

But as the upper lateral systems of through bridges and the lower lateral systems of deck bridges are not affected by the wind pressure upon the train, the author considers that empty spans from 200 to 250 feet in length should be proportioned for forty-five pounds per square foot, and all greater spans for forty pounds.

In reality these figures have not been exactly adhered to in making the designs for this treatise, because the author has considered it better to reduce the intensities of wind pressure gradually than to change them suddenly by decrements of five pounds per square foot.

Theodore Cooper Esq. C. E., the author of the best American bridge specifications, provides for a wind pressure of 150 pounds per lineal foot for upper lateral bracing in through bridges and lower lateral bracing in deck bridges. This is a rather small allowance for a country visited annually by typhoons. In preparing Table XIII the author used 150 pounds for spans of 100 feet and under, from that to 200 pounds for spans between 100 and 200 feet and from 200 to 240 pounds for spans from 200 to 300 feet as the pressures per lineal foot for upper lateral bracing. The pressures per lineal foot on trusses only for the lower lateral systems were calculated to be from 290 pounds for spans of 100 feet to 320 pounds for those of 300 feet for empty bridges; and from 170 pounds for spans of 100 feet to 240 pounds for those of 300 feet for bridges covered by the moving load.

The pressures per lineal foot upon the upper lateral systems with an intensity of thirty pounds are about 90 pounds for spans of 100 feet and under, from 90 to 130 pounds for spans between 100 and 200 feet, and from 130 to 180 pounds for spans between 200 and 300 feet.

The method of calculating these pressures was fully explained in the last chapter. A portion of the leeward truss is protected by the train, but no deduction should be made on this account, because the surfaces of the channels, being concave towards the wind, tend to increase the intensity of pressure: indeed it is well in figuring areas to allow an inch or two of extra width to compensate for this concavity. In this particular system of bridges the effect of wind pressure on bottom chord tension need not be considered in spans under one hundred and thirty feet in length.

Wind loads upon empty bridges are treated as moving, for it is possible for one part of a bridge to be protected while the remainder is exposed; besides the centre of a whirlwind has a motion of translation which would cause the pressure to really act as a moving load. This method of treatment affects principally the lateral rods and

struts near the middle of the span. The pressure upon the train is undoubtedly a moving load, but the coexisting pressure upon the trusses must be treated as static; for it would be highly improbable that a maximum wind and a train could advance together and with the same velocity upon a bridge.

CHAPTER VIII.

STRESSES IN TRUSSES.

The causes of stresses in trusses are the following;

1° uniform live load
2° dead load
3° engine excess
4° wind pressure directly
5° wind pressure indirectly
6° curvature of track

The uniform live load and engine excess produce tension in the bottom chords, main diagonals and counters, and compression in the top chords posts and batter braces.

The dead load affects similarly the chords, posts and main diagonals, but does not affect the counters, or rather it tends to diminish the stresses on the counters. The direct stresses due to wind pressure are compression on upper and lower windward chords, and tension on upper and lower leeward chords.

The indirect stresses due to wind pressure are equivalent to those produced by increasing the dead load on the leeward truss: they will be called "transferred load stresses." The wind also relieves the dead load on the windward truss: the stresses due to the difference between the dead load on this truss and the reduction will be called "reduced dead load stresses."

The curvature of the track produces a centrifugal force which acts only on either the lower or upper chords, according to whether the bridge be through or deck: it affects also the corresponding lateral system, and the vertical sway bracing of deck bridges. The combination of all these stresses is a little complicated, especially as the direct and indirect stresses due to wind pressure may be subdivided into the two cases, first when the bridge is empty, and second when it is partially or wholly covered by the moving load. The following table will facilitate the comprehension of the effects of the various loads upon the different members, T standing for tension and C for compression.

No.	Cause of Stresses.	Wind. Top Chd.	Leew.d Top Chd.	Wind. Bot. Chd.	Leew.d Bot. Chd.	Wind. Posts	Leew.d Posts	Wind. M. Diag.	Leew.d M. Diag.	Wind. Coun-ters	Leew.d Coun-ters	Wind. H. Vert.	Leew.d H. Vert.	Wind. B. Brace	Leew.d B. Brace
1	Uniform Live Load.	C	C	T	T	C	C	T	T	T	T	T	T	C	C
2	Dead Load.	C	C	T	T	C	C	T	T	—T or —	—T or —	T	T	C	C
3	Engine Excess.	C	C	T	T	C	C	T	T	T	T	T	T	C	C
4	Wind Pressure Bridge Empty.	C	T	C	T	—	—	—	—	—	—	—	—	—	—
5	Wind Pressure Bridge Loaded.	C	T	C	T	—	—	—	—	—	—	—	—	—	—
6	Transferred Load Bridge Empty.	—C	C	—T	T	—C	C	—T	T	—	—	—T	T	—C	C
7	Transferred Load Bridge Loaded.	—C	C	—T	T	C	C	—T	T	—T	T	—T	T	—C	C
8	Reduced Dead Load. Bridge Empty.	C	—	T	—	C	—	T	—	—	—	T	—	C	—
9	Reduced Dead Load. Bridge Loaded.	C	—	T	—	C	—	T	—	—	—	T	—	C	—
10	Centrifugal Force. Thro. Bridges.	—	—	C or T	T or C	—	—	—	—	—	—	—	—	—	—
11	Centrifugal Force. Deck Bridges.	C or T	T or C	—	—	—	—	—	—	—	—	—	—	—	—
12	Bending.	—	—	—	—	C	C	—	—	—	—	—	—	C	C

To properly apply this table one must distinguish between a probable or ordinary combination and an improbable or extraordinary combination of stresses; and must recognize which stresses *may* and which stresses *must* exist together. There are two distinct conditions, first when the bridge is empty, and second when it is wholly or partially covered by the moving load.

Distinction must be made between the stresses in certain members of deck bridges and those in the corresponding members of through or pony truss bridges.

Again any particular member may receive its greatest stress when it belongs to the windward or to the leeward truss. Keeping these facts in view let us analyze the table.

First when the bridge is empty we are concerned with the horizontal lines numbered 2, 4, 6, 8, and 12; and, when it is loaded, with the lines 1, 2, 3, 5, 7, 9, 10 or 11 and 12.

Commencing with the top chord, first when the bridge is empty, we have 2°, 4,° 6° and 8° acting but the latter is a combination of 2° and 6° so need not be considered. We see that for both windward and leeward chords there are both C and T or—C acting simultaneously, for 4° and 6° cannot act independently. On this account and because the effect of 1° and 3° is so great we conclude that the top chord stresses when the bridge is empty need not be considered.

When the bridge is loaded 1°, 2°, 3°, 5°, 7° and 9° act in through or pony truss bridges and for deck bridges 11° may also act. In deck bridges 5° has a much greater effect than in through or pony truss bridges, for it then includes the wind pressure upon the train.

As before 9° may be omitted as it is a combination of 2° and 7°: this leaves for through and pony truss bridges 1°, 2° and 3° as the ordinary loading and 1°, 2°, 3°, 5° and 7° as the extraordinary loading. In the latter it will be noticed that for both windward and leeward chords we have both C and T or—C acting simultaneously; for 5° and 7° must act together. On this account and because of the great effect of 1° and 3° compared with 5°, it is evident that 1°, 2° and 3° are the loading to be considered.

In general as will have been noticed in reading chapter VI, extraordinary loads need not be considered unless their effects exceed those of ordinary loads by about fifty per cent.

In deck bridges it may be possible, though it is not probable, that the combined effect of 1°, 2°, 3°, 5° and 7° may be so much greater than that of 1°, 2° and 3° that the method of proportioning given in chapter VI will have to be adopted. If there be curvature in the track 11° will always exist with both 1°, 2° and 3°, and with 1°, 2°, 3°, 5° and 7°. Its effect may be considered as an increment of that of 1° and 3° in proportioning by the method of chapt. VI, but it is to be noticed that it need increase the sectional area of the chord on the concave side of the track only. In 11° it is seen that the effect may be either C or T, but C must be taken, as the wind may act in either direction and consequently both with and against the centrifugal force.

Although the centrifugal force really reduces the stress on one chord, it should not be assumed so to do; for the trains do not necessarily pass over the bridge at their maximum velocity.

Passing to the bottom chord we may conclude at once both from the table and from our general knowledge of bridges, that in through and pony truss bridges the stresses will be greatest when the bridge is loaded. When, however, the bridge is empty we have acting upon the windward chord 2°, 4° and 6°, or what is the same thing 4° and 8°, the former producing C and the latter T; and as in most cases for the bridges dealt with in this treatise the former exceeds the latter, the bottom chords should be proportioned to resist a compression of C_4—T_8, the subscripts denoting the horizontal line to which the stresses belong.

When the bridge is loaded, 1°, 2°, 8° and perhaps 10° act together as an ordinary load while 1°, 2°, 3°, 5°, 7° and perhaps 10° act together as an extraordinary load. Where the latter exceeds the former by more than fifty per cent, the chords should be proportioned to resist the stresses produced by the second loading using an intensity of working stress of 7. 5 tons, but otherwise the stresses due to the first loading are to be taken, using an intensity of working stress of 5 tons.

It must be noticed that T_{10} affects the sizes of the chord sections on the convex side of the track only, and that, although given as either C or T, T must be taken, as the wind acts in either direction.

In deck bridges 11° cannot act, and as the effect of 5° is small compared with that of 1° and 8°, the loading to be considered is that of 1°, 2° and 8°. For the empty bridge C_4 is not quite so liable to exceed T_8 as in the case of through or pony truss bridges, because the wind load carried by the lower lateral system is less.

Passing to the posts we may conclude immediately that they take their greatest stresses when the bridge is partially loaded, in which case 1°, 2°, 8° give the ordinary and 1°, 2°, 8°, 7° and 12° the extraordinary loadings. Except in the case of very light posts the latter loading need not be considered; but when it is, the post must be proportioned as directed in Chapter VI.

Passing to the main diagonals we have 1°, 2° and 8° as the ordinary loading, and 1°, 2°, 8° and 7° as the extraordinary loading. It is self evident that the latter need not be considered.

The same remark applies to the counters and hip verticals.

Finally passing to the batter braces we may immediately conclude that the stresses existing when the bridge is empty need not be considered, hence the ordinary loading will be for through bridges, pony truss bridges and deck bridges with straight track 1°, 2° and 8°, and the extraordinary loading 1°, 2°, 8°, 7° and 12°. It is highly improbable that the latter loading need ever be considered. For deck bridges with curved track the ordinary loading will be 1°, 2°, 8° and 12°, the bending being produced by the centrifugal force; and the extraordinary loading 1°, 2°, 8°, 7° and 12°, the bending being produced by wind pressure and centrifugal force acting together. As before, the ordinary load is the one usually to be provided for; if not, the proportioning must be done as directed in Chapter VI.

To recapitulate; in through and pony truss bridges the top chords must be proportioned to resist the uniform live load, dead load and engine excess stresses, and in deck bridges to resist either the same, together with curvature stresses, if any, or the combined live load, dead load, engine excess, wind and, if necessary, curvature stresses. The bottom chords for through and pony truss bridges are to be proportioned to resist a compression equal to the difference between the wind stresses and the reduced dead load stresses when the bridge is empty; also either the live load, dead load engine excess and, if necessary, curvature stresses, or a combination of the same with the wind load tension.

The bottom chords for deck bridges have to resist a compression equal to the difference between the wind stresses and the dead load stresses when the bridge is empty, and a tension due to the live load, dead load and engine excess.

Posts are to be proportioned to resist either the live load, dead load and engine excess stresses, or these combined with the transferred load and bending stresses, the latter being produced by a wind pressure of thirty pounds per square foot.

Main diagonals, counters and hip verticals have to resist the live load, dead load and engine excess stresses; and batter braces either the three last mentioned for through and pony truss bridges, or the same combined with bending stresses due to curvature for deck bridges; and a combination of live load, dead load, engine excess, transferred load and bending stresses for either through or deck bridges, the bending being due to the wind pressure alone in through bridges or to either the wind pressure alone or the combined wind pressure and centrifugal force in deck bridges.

As is customary in figuring stresses, uniformly distributed loads will be considered as concentrated at the panel points, and the half panel load at each end of the span is not supposed to produce any stress in any member of the truss.

In order to facilitate the calculation of stresses, the following nomenclature will be adopted.

n = number of panels in span.

l = length of each panel.

d = depth of truss from centre to centre of chords.

b = perpendicular distance between central planes of trusses.

θ = inclination of diagonal ties to the vertical in single intersection trusses.

α = inclination of short diagonals to the vertical in double intersection trusses.

β = inclination of long diagonals to the vertical in double intersection trusses.

θ' = the angle whose tangent is $\frac{l}{b}$.

W = live panel load on one truss (*i. e.* one half the product of the uniform live load per lineal foot by the panel length).

W_1 = panel dead load on one truss.

W' = that portion of W_1 concentrated at the upper panel point.

W_2 = wind pressure concentrated at a windward and leeward panel point when the bridge is empty.

W_3 = wind pressure concentrated at a windward and leeward panel point when the bridge is loaded.

W_4 = load transferred from windward panel point to leeward panel point, when the bridge is empty.

W_5 = load transferred from windward panel point to leeward panel point, when the bridge is loaded.

$W_6 = W_1 - W_4$ = reduced panel dead load on windward truss, when the bridge is empty.

W_7 = panel load due to centrifugal force produced by cars: this is for convenience supposed to be equally divided between the panel points of opposite trusses, although this method may produce a small increase of stress upon the lateral struts above those which will actually exist.

E = panel engine excess on one truss.

E_1 = panel load due to centrifugal force produced by engine excess: it is concentrated as is W_7.

The first step in calculating stresses is to fill out one of the following tables of data

Single Intersection.	Double Intersection Even Number of Panels.	Double Intersection Odd Number of Panels.
$n =$	$n =$	$n =$
$l =$	$l =$	$l =$
$d =$	$d =$	$d =$
$b =$	$b =$	$b =$
diag. =	short diag. =	short diag. =
sec. θ =	long diag. =	long diag. =
tan. θ =	sec. α =	sec. α =
tan. θ' =	tan. α =	tan. α =
W =	sec. β =	sec. β =
W_1 =	tan. θ' =	tan. θ' =
W' =	W =	W =
W'' =	W_1 =	W_1 =
W_2 =	W' =	W' =
W_3 =	W'' =	W'' =
W_4 =	W_2 =	W_2 =
W_5 =	W_3 =	W_3 =
W_6 =	W_4 =	W_4 =
W_7 =	W_5 =	W_5 =
E =	W_6 =	W_6 =

$E_1 =$

$\frac{1}{n} W =$

$\frac{1}{n} W \sec. \theta =$

$W_1 \sec. \theta =$

$\frac{1}{2} W_1 \sec. \theta =$

$W'' \tan. \theta =$

$\frac{1}{2} W'' \tan. \theta =$

$\frac{1}{n} E =$

$\frac{1}{n} E \sec. \theta =$

$\frac{1}{n} E \tan. \theta =$

$W_2 \tan. \theta' =$

$\frac{1}{2} W_2 \tan. \theta' =$

$W_3 \tan. \theta' =$

$\frac{1}{2} W_3 \tan. \theta' =$

$W_4 \tan. \theta =$

$\frac{1}{2} W_4 \tan. \theta =$

$W_5 \tan. \theta =$

$\frac{1}{2} W_5 \tan. \theta =$

$W_6 \tan. \theta =$

$\frac{1}{2} W_6 \tan. \theta =$

$W_7 \tan. \theta' =$

$\frac{1}{2} W_7 \tan. \theta' =$

$\frac{1}{n} E_1 \tan. \theta' =$

$W_7 =$

$E =$

$E_1 =$

$\frac{1}{n} W =$

$\frac{1}{n} W \sec. \alpha =$

$W_1 \sec. \alpha =$

$\frac{1}{2} W_1 \sec. \alpha =$

$\frac{1}{n} W \sec. \beta =$

$W_1 \sec. \beta =$

$\frac{1}{2} W_1 \sec. \beta =$

$W'' \tan. \alpha =$

$\frac{1}{2} W'' \tan. \alpha =$

$\frac{1}{n} E =$

$\frac{1}{n} E \sec. \alpha =$

$\frac{1}{n} E \sec. \beta =$

$\frac{1}{n} E \tan. \alpha =$

$W_2 \tan. \theta' =$

$\frac{1}{2} W_2 \tan. \theta' =$

$W_3 \tan. \theta' =$

$\frac{1}{2} W_3 \tan. \theta' =$

$W_4 \tan. \theta =$

$\frac{1}{2} W_4 \tan. \theta =$

$W_5 \tan. \theta =$

$\frac{1}{2} W_5 \tan. \theta =$

$W_6 \tan. \theta =$

$\frac{1}{2} W_6 \tan. \theta =$

$W_7 \tan. \theta' =$

$\frac{1}{2} W_7 \tan. \theta' =$

$\frac{1}{n} E_1 \tan. \theta' =$

$W_7 =$

$E =$

$E_1 =$

$\frac{1}{n} W =$

$\frac{1}{n} W_1 =$

$\frac{1}{n} W \sec. \alpha =$

$\frac{1}{n} W_1 \sec. \alpha =$

$\frac{1}{n} W \sec. \beta =$

$\frac{1}{n} W_1 \sec. \beta =$

$\frac{1}{n} W'' \tan. \alpha =$

$\frac{1}{n} E =$

$\frac{1}{n} E \sec. \alpha =$

$\frac{1}{n} E \sec. \beta =$

$\frac{1}{n} E \sec. \alpha =$

$W_2 \tan. \theta' =$

$W_3 \tan. \theta' =$

$W_4 \tan. \theta =$

$W_5 \tan. \theta =$

$W_6 \tan. \theta =$

$W_7 \tan. \theta' =$

$\frac{1}{n} E \tan. \theta' =$

The value of W_1 may be taken as $\frac{1}{3} W_1$ for through bridges, $\frac{1}{2} W_1$ for pony truss bridges and $\frac{2}{3} W_1$ for deck bridges: in short through spans this causes a small error on the side of safety, and in short deck spans a correspondingly small error on the side of danger. For the latter it might be well to make $W' = \frac{3}{4} W_1$

Having filled out the table of data, the next step is to draw a skeleton diagram large enough to contain all the stresses and sections. It is not necessary that the diagram be drawn to scale; but the ratio of panel length to depth of truss on the diagram, for the sake of appearance, should not vary too greatly from the ratio of the actual values of these dimensions. A panel length of an inch and a half, and a

depth of two inches and a half, are about as small dimensions as will be found convenient.

At each lower panel point write lightly in pencil, so that it can be afterwards erased, the number of the panel point, beginning with zero at the right-hand end of the span.

It is well known, and will be accepted here without proof, that the greatest stresses in the chords and batter braces occur when the bridge is entirely covered by the moving load; that the greatest stress in any diagonal exists when the live load extends to its foot from that end of the bridge towards which the diagonal points in a *downward* direction; that the greatest stress in any post in through and pony truss bridges occurs when the main diagonal (or, if there be none, when the heaviest counter) attached to its upper end (or in deck bridges when that attached to its lower end) receives its greatest stress; and that the two diagonals of a panel cannot at the same time be subjected to the same kind of stress, excepting, of course, the initial tension.

It is apparent that when the greatest stresses in all the diagonals sloping upward in one direction, and in all the posts and chord panels on one side of the central plane, are found, the greatest stresses in the diagonals sloping in the opposite direction, and in the posts and chord panels on the other side of the central plane, can be immediately written. This fact is so well known, that, in making a diagram of stresses, it is usual to write the stresses on only one-half of the members of the truss.

First let us investigate the stresses due to the uniform live load and the dead load, and begin with a single-intersection through or pony truss bridge.

The greatest stress in any diagonal sloping upward from right to left can be found by the formula

$$T = \frac{n'}{2}(n'+1)\frac{W}{n}\sec\theta + \left(n' - \frac{n-1}{2}\right)W_1 \sec\theta,$$

where n' is the number of the panel point at the foot of the diagonal. This formula is applicable to counters as well as to main diagonals. If the stress should come out negative, it shows that no counter is needed in the panel considered. It is also applicable to the batter brace by putting $(n - \mathrm{I})$ for n'.

The stress in any post can be found by the formula.

$$C = \frac{n'(n'-1)}{2}\cdot\frac{W}{n} + \left(n' - \frac{n+1}{2}\right)W_1 + W',$$

where n' (not less than $\frac{n}{2}$) is the number at the foot of the post.

The stress in any panel of the top chord is given by the formula

$$C = \frac{n'(n-n')}{2}W'' \tan\theta,$$

where n' (not greater than $\frac{n}{2}$) is the number at the end of the panel nearest to the centre of the bridge.

The stress in any panel of the bottom chord, except the one at the end of the

span, is given by the formula

$$T = \frac{(n'-1)(n-n'+1)}{2} W'' \tan \theta,$$

n' having the same value as in the last formula. For the end panel, the stress is the same as for the second panel.

As the values of $\frac{W}{n'}$, $\frac{W}{n}$ sec θ, and W_1 sec θ, are given in the table of data, the substitution in these formulas is a very simple matter.

The stress in the hip verticals of any through or pony truss is equal to one half of the floor beam load including its own weight plus the weight of a panel length of the lower chord. It is not necessary to calculate this stress for single track bridges, as the sections required for and sizes of hip verticals for all practical cases are given in Table VII.

In deck bridges the hip verticals sustain only the weight of a panel length of the bottom chord and lower lateral system so that two one inch rods will be sufficient.

The only other difference between the stresses in a deck bridge and those in a corresponding through bridge will be in the posts, the stresses for which are to be found by letting the live load extend from the farthest end of the bridge to the top of the post; so that the post will no longer take its greatest stress with the main diagonal attached to its top, but with the one attached to its foot.

The formula for post stresses in single-intersection deck bridges is, therefore,

$$C = \frac{n'(n'+1)}{2}\left(\frac{W}{n}\right) + \left(n' - \frac{n+1}{2}\right) W_1 + W'.*$$

Some engineers may object to using formulas for figuring stresses: if so, the following method will give the same results for single-intersection bridges.

Pass a vertical plane through the middle point of the bottom chord: all the dead loads to the right of this plane may be considered to go to the right-hand pier, and all to the left of the plane to the left-hand pier. Should there be a post at the middle of the bridge, the weight at the foot is to be considered as halved, one-half going

* In "The Designing of Ordinary Iron Highway Bridges" this formula is given as

$$C = \frac{n'(n'-1) + 2n}{2}\left(\frac{W}{n}\right) + \text{\&c.}$$

This was obtained under the supposition that the load W on top of the post passes down the post before being separated into the portions which go to the right and left, but the author has come to the conclusion that the portion, which passes to the farther end of the span, goes down the main diagonals as compression or in reality as a reduction of tension. The error in the last formula is upon the side of safety and varies between zero and $\frac{W}{2}$: moreover its greatest effect is upon the posts at and near the middle of the span, which posts in many cases have necessarily greater sections than the stresses call for.

Besides the load comes more quickly upon the posts of deck bridges than upon those of through bridges, so some engineers might prefer to use the less correct formula.

to each pier. Then the stress in any main diagonal of the left-hand half of the bridge is to be found by commencing at the right-hand end, and adding the numbers at the panel points until the foot of the diagonal considered is reached, multiplying the sum by $\frac{1}{n}$ $W \sec \theta$, and to the product adding the number of panel dead loads between the central plane and the panel point at the foot of the diagonal considered (including the one at this point) multiplied by $W_1 \sec \theta$.

For instance, in a ten-panel bridge, the stress in the end main diagonal, the number at its foot being eight, will be

$$(1+2+3+\text{ etc.} \ldots +8)\frac{W \sec \theta}{10}+(\tfrac{1}{2}+1+1+1)W_1 \sec \theta.$$

The stress in a counter on the right-hand half of the bridge will be found by adding the numbers at the panel points until the foot of the counter considered is reached, multiplying the sum by $\frac{1}{n}$ $W \sec \theta$, and from the product subtracting the dead-load-stress of the main diagonal which crosses the counter. Thus, in the ten-panel bridge, the stress in the second counter from the centre in the right-hand half of the spans, or the one at the foot of the third panel point, is

$$(1+2+3)\frac{W \sec \theta}{10}-(\tfrac{1}{2}+1)W_1 \sec \theta.$$

The greatest stress in any post of a through or pony truss bridge is found by adding W' to the vertical component of the greatest stress in the main diagonal attached to its upper end, or $\frac{n'}{n}$ W + W' in case of a deck bridge, thus in the assumed bridge, which may be taken as a through one, the stress in the first post from the left-hand end, or the one at the eighth panel point, is

$$(1+2+3+\text{ etc.} \ldots +7)\frac{W}{n}+(\tfrac{1}{2}+1+1)\ W_1+W'$$

For the case of a middle post, the stress in one of the counters at the upper end must be substituted for that of the main diagonal; thus, in the same bridge, the stress in the middle post is

$$(1+2+3+4)\frac{W}{n}-\tfrac{1}{2}\ W_1+W'.$$

The stresses in the chords are to be found by the following method:—

Pass a plane through the foot of the post at or nearest to the middle of the truss, and take the centre of moments at this foot. From the moment of the reaction at the nearest end of the bridge subtract the sum of the moments of the panel loads (W'') lying between the centre of moments and this end, and divide the difference by the depth of the truss. The result will be the stress in the panel of the top chord nearest the centre of the bridge: it will be some multiple of $W'' \tan \theta$.

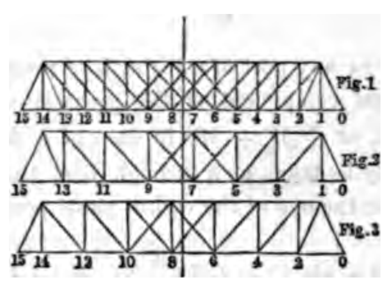
The stress in the panel of the bottom chord immediately below will be equal to the one found, less the horizontal component of the *main diagonal* of the panel, when

the bridge is covered by the moving load. This horizontal component will be zero for a truss with an odd number of panels, and $\frac{1}{2}$ W' tan θ for a truss with an even number of panels.

The stress in the next panel of the bottom chord towards the nearest end of the bridge is found by subtracting from the one already determined the horizontal component of the stress in the *main diagonal* at the panel point between the two panels considered; the bridge, as before, being fully loaded. This component is a multiple of W'' tan θ. In this way can be found all the stresses in the panels of the bottom chord, the correctness of the work being checked by seeing if the stress in the end panel be equal to the re-action multiplied by tan θ. If so, the remaining upper-chord stresses may be at once written by inspection; for the stress in the nth panel of the top chord, counting from the nearest pier or abutment, and supplying the missing panel at the end, is numerically equal to that in the $(n + 1)$th panel of the bottom chord.

Next let us consider the double-intersection truss.

The formulas for this case are so complicated that it is better not to employ them. The simplest method is to draw a skeleton diagram, and number the panel points, as in the single-intersection truss. The double-intersection truss really consists of two trusses, as may be seen in the accompanying diagram.



Such a division is necessary in order to calculate the chord stresses when the truss contains an odd number of panels. This is accomplished by finding by the method of moments already explained, the chord stresses in each of the trusses shown in Figs. 2 and 3, and then combining them. Thus the stress in panel 9-10 of the lower chord in Fig. 1 is equal to that in panel 9-11 of Fig. 2, plus that of panel 8 – 10 of Fig. 3.

The live-load stress in any diagonal sloping upward from right to left is found by noting whether the number at its foot be odd or even, then taking the sum of the odd or even numbers, from one or two up to the number at the foot of the diagonal, and multiplying the sum by $\frac{W}{n}$ sec α, or $\frac{W}{n}$ sec β, as the case may be.

The stress due to the dead load is found by taking the sum of the same numbers, and from it subtracting the sum of the odd or even numbers from one or two up to $(n - n' - 2)$, where n is the number of panels in the span, and n' is the number at the foot of the diagonal considered. Whether the odd or even numbers should be

taken can be ascertained by following out towards the left the system to which the diagonal belongs: if the system contain the short diagonal at that end, then the even numbers are to be taken, otherwise the odd ones.

The difference thus found, multiplied by $\frac{W_1 \sec \alpha}{n}$, or $\frac{W \sec \beta}{n}$, as the case may be, will give the dead-load stress in the diagonal. Thus, in the diagram, the dead-load stress in the main diagonal at the panel point 10 is

$$[(2+4+\text{ etc. }+10)-(1+3)]\frac{W_1 \sec \beta}{n}$$

As in the case of the single intersection, the stress in a main diagonal is equal to the sum of the live and dead load stresses; that in a counter, to the difference between its live-load stress and the dead-load stress of the main diagonal crossing it at the middle of its length; that in a post of a through or pony truss bridge, by the sum of W' and the vertical component of the greatest stress in the main diagonal (or, if there be none, that in the principal counter) attached to its upper end. As the batter braces belong to both systems of triangulation, their stresses are the sum of the stresses found by each system, or by the formula

$$C=[1+2+3+\text{etc.}\ldots+(n-1)]\frac{(W+W_1)\sec \alpha}{n}.$$

If the number of panels be even, the calculation for the dead-load stresses may be much simplified by counting the number of panel points on the system considered lying between the central plane and the panel point at the foot of the diagonal, including the latter, remembering that the load at the middle panel is halved, and multiplying the result by $W_1 \sec \alpha$, or $W_1 \sec \beta$.

The finding of the chord stresses is also simplified when there is an even number of panels; for they can then be calculated by the method explained for the single-intersection truss.

To find the stress in a post of a double-intersection deck-bridge add together, $\frac{n'}{n}W$, W' and the vertical component of the greatest stress in the principal diagonal attached to its upper end.

The stresses in hip verticals are found in the same manner as in single-intersection trusses.

In every double-intersection truss, there is necessarily a little ambiguity; for it is possible that the whole of the load concentrated at the first panel point does not travel by the system of odd numbers; but this ambiguity is a matter of small moment. Next let us investigate the stresses due to engine excess. In every case it is better to assume the worst possible distribution of the loads in calculating the stresses produced thereby. Thus in the double intersection truss the greatest effect upon the web members is found by placing a car between the two locomotives, while that upon the top chords usually requires the locomotives to be coupled together. Again, in both single and double intersection trusses the greatest chord stresses are obtained by placing cars in front of and behind the locomotives.

It may appear to some readers that the effect of the engine excess on the chords is exaggerated, in that the worst position is chosen, but it must be remembered that engines are occasionally preceded and followed by cars: this may not occur often in Japan, but it is not unusual in America.

The greatest stresses in the top chords exist when the leading engine excess is at that end of the panel considered lying nearest the centre of the span. The stress is given by the equation

$$C = (2\,n' - 1)\,(n - n')\,\frac{E}{n}\tan\theta$$

for single intersection trusses where the panel length is not less than eighteen feet, or by the equation

$$C = 2\,(n' - 1)\,(n - n')\,\frac{E}{n}\tan\theta$$

when the panel length is less than eighteen feet. For double intersection trusses the formula is

$$C = [\,n'\,(n - n') + (n' - 1)\,(n - n' + 1)\,]\,\frac{E}{n}\tan\alpha$$

n' (never less than $\frac{n}{2}$) being the number of the panel point where the leading engine excess rests. This last formula fails to give the greatest stress in the first panel of the top chord of double intersection trusses, when the number of panels exceeds eleven, but as the error is a very small one on the side of danger, and as there are seldom more than eleven panels, the formula may be taken as correct enough for all cases. The reason for the existence of the error is that in double intersection trusses having more than eleven panels, the end length of the top chord takes its greatest stress when the engine excesses are separated by one panel point.

Where the number of panels is odd the engine excess stress in the middle panel of the top chord is the same as that in the consecutive panels, found by making $n' = \frac{n+1}{2}$ in the formula.

The stress in the first and second panels of the bottom chord in either single or double intersection trusses is given by the formula

$$T = (2n - 3)\,\frac{E}{n}\tan\theta \qquad [\text{ or } \tan\alpha\,]$$

except for the case of single intersection trusses with panels less than eighteen feet in length when it is given by

$$T = 2\,(n - 2)\,\frac{E}{n}\tan\theta$$

In single intersection trusses, where the panel length is not less than eighteen feet, the stress in any other panel of the bottom chord is given by the equation

$$T = (2\,n' - 1)\,(n - n')\,\frac{E}{n}\tan\theta$$

and where it is less than eighteen feet, by the equation

$$T = 2\,(n' - 1)\,(n - n')\,\frac{E}{n}\tan\theta$$

where n' (greater than $\frac{n}{2}$) is the number at that end of the panel considered lying next to the nearer end of the span; for the bottom chord in these cases takes its greatest stress when the leading engine excess is at that end of the panel considered farthest from the centre of the span.

The last formula fails for the case of a middle panel, when the number of panels is odd. In this case the stress is given by the formula

$$T = \left(\frac{(n-1)^2}{2} - 1 \right) \frac{E}{n} \tan \theta$$

In double-intersection trusses the stress in any other panel of the bottom chord than the first or second is given by the equation

$$T = 2\,(n'-1)\,(n-n')\,\frac{E}{n} \tan \alpha$$

where n' (not less than $\frac{n}{2}+1$) is as before the number at the end of the panel farthest from the zero end of the span. This limitation excludes the calculating by this formula of the stress in the middle panel of every truss having an odd number of panels; but such a calculation is unnecessary, for the greatest engine excess stress in the middle panel is equal to that in the next consecutive panel. From this formula it is apparent that all the panels of the bottom chord, excepting the first and second from the ends, take their greatest stresses when the engine excesses are separated by a panel point. It was seen that such is rarely the case with the top chord panels.

The engine excess stress in any main diagonal or counter of a single intersection truss, in which the panel length is not less than eighteen feet, is given by the equation

$$T = (2\,n' - 1)\,\frac{E}{n} \sec \theta\,;$$

and, if the panel length be less than eighteen feet, it is given by the equation

$$T = 2\,(n'-1)\,\frac{E}{n} \sec \theta$$

For double intersection trusses the corresponding formula is

$$T = 2\,(n'-1)\,\frac{E}{n} \sec \alpha, \text{ (or } \sec \beta\text{)}$$

For single intersection through or pony truss bridges where the panel length is not less than eighteen feet, the engine excess stress on any post is given by the formula

$$C = (2\,n' - 3)\,\frac{E}{n}\,;$$

and, where the panels are less than eighteen feet, it is given by the formula

$$C = 2\,(n' - 2)\,\frac{E}{n}$$

For double intersection trusses the formula is

$$C = 2\,(n' - 3)\,\frac{E}{n}$$

For deck bridges the corresponding formulæ are respectively

$$C = (2\,n' - 1)\,\frac{E}{n}$$

$$C = 2\,(n' - 1)\,\frac{E}{n}$$

and $$C = 2\,(n' - 1)\,\frac{E}{n}$$

It seems almost needless to say that the engine excess, being merely a conventional load, should not be used in finding stresses in hip verticals; but that the method previously given takes into account the panel engine *load* though not the engine *excess*. The use of all the previous formulæ may always be avoided by employing Tables III and IV, which give the greatest stresses due to the uniform live load, the dead load and the engine excesses on the various members of trusses for all practical cases.

The wind stress on any windward panel of either top or bottom chord when the span is empty is given by the equation

$$C = \frac{n'\,(n - n')}{2}\,W_2 \tan\theta'$$

and that on any leeward panel by the equation

$$T = \frac{(n'+1)\,(n - n' - 1)}{2}\,W_2 \tan\theta'$$

where n' (not less than $\frac{n}{2}$) is the number at that end of the panel considered which lies nearest the centre of the span. The corresponding wind stresses when the span is covered by the moving load can be found by substituting W_3 for W_2 in the last two equations. It is obvious that these equations apply to through, pony truss and deck bridges. Their use may be avoided by employing Table V, which gives the wind and curvature stresses for all practical cases.

To ascertain the amount of the transferred load when the bridge is empty.

Let h = vertical distance between horizontal plane of shoe plates and the centre of gravity of the vertical projection of the trusses

and p = total wind pressure per lineal foot of bridge on both trusses.

Then the overturning moment will be $p\,h$, which is a couple formed by a pressure p and an equal horizontal reaction $\frac{P}{S}$, where P is the total horizontal reaction of the the four shoes and S the length of span. This must be resisted by another couple of equal but opposite moment. The forces of this couple can only be a

released weight w_4 upon the windward truss and an increased weight of the same amount upon the leeward truss, which will give the equation

$$p\,h = w_4\,b$$

$$\text{or } w_4 = \frac{ph}{b}$$

An approximate value for the overturning moment can be found by calling p' the pressure per lineal foot of span concentrated upon the upper lateral system, then

$$p'\,d = w_4\,b$$

$$\text{and } w_4 = \frac{p'd}{b}$$

The latter value of w_4 is not so exact as the former, but is much more easily calculated.

The transferred panel load W_4 is of course equal to $w_4\,l$, and the reduced panel oad $W_6 = W_1 - W_4 = W_1 - w_4\,l$.

The following table gives approximate values for w_4 for the single track through bridges of this treatise

Span	w_4	Span	w_4	Span	w_4
70'	160	160'	290	220'	440
80'	180	170' S. I.	300	230'	460
90'	210	170' D. I.	340	240'	480
100'	230	180' S. I.	320	250'	500
110'	240	180' D. I.	360	260'	520
120'	250	190' S. I.	340	270'	540
130'	260	190' D. I.	380	280'	560
140'	270	200'	400	290'	580
150'	280	210'	420	300'	600

The value of w_5, the transferred load per lineal foot of span, may be calculated as follows.

The wind pressure of 240 pounds per lineal foot acts with a leverage of about eight feet, producing an overturning moment of $8 \times 240 = 1920$ foot pounds. If in the equation

$$w_4 = \frac{p'd}{b}$$

we substitute w_5 for w_4 and p'' (the pressure per lineal foot of span upon the upper lateral system, when the wind pressure is thirty pounds per square foot) for p', and add 1920 foot pounds to the moment, we will have

$$w_5 = \frac{p''d + 1920}{b}$$

From this equation has been prepared the following table.

Span	w_5	Span	w_5	Span	w_5
70′	240	160′	300	220′	410
80′	250	170′ S. I.	310	230′	440
90′	260	170′ D. I.	330	240′	470
100′	270	180′ S. I.	320	250′	480
110′	275	180′ D. I.	340	260′	500
120′	280	190′ S. I.	330	270′	515
130′	285	190′ D. I.	350	280′	530
140′	290	200′	370	290′	545
150′	295	210′	390	300′	560

It may be noticed that these values of w_5 do not agree exactly in all cases with those used in preparing the diagrams of stresses which accompany this treatise. Both values are merely approximate, but those in the table are preferable.

The reason for the disagreement is that the area per lineal foot of span opposed to the wind had first to be assumed and afterwards checked. As the disagreement varies from zero to only seven per cent of the transferred load, and as the latter is but a small portion of the total load, it is not worth while to correct the diagrams of stresses.

The formulae for the stresses due to W_5 and W_6 are identical with those for W_1: they can either be obtained from the previous part of this chapter, or from one of Tables III and IV.

To calculate the values of W_7 and E_1

let v = maximum velocity of train in feet per second
g = acceleration of gravity
r = radius of the curve

$$w = \frac{W}{l} = \text{live load per foot of span}$$

and $w_7 = \dfrac{W_7}{l}$ = centrifugal force per lineal foot due to car load

then $$w_7 = \frac{w}{g} \cdot \frac{v^2}{r}$$

For sixty miles per hour $v = 88$; and $g = 32.2$, therefore

$$w_7 = \frac{w\,(88)^2}{32.2\,r} = 240\,\frac{w}{r},$$

from which it is evident that, in order to make the centrifugal force have the same effect as the live load, the radius of the curve must be 240 feet. Generally it is not much less than 1,000 feet, consequently the centrifugal force seldom produces stresses one fourth as great as those due to the live load. In a similar manner we can establish the equation

$$E_1 = 240\,\frac{E}{r}$$

The formulae for the stresses due to W_7 are identical with those due to W_2 and W_3.

The formula for the concave side chord stresses due to E_1, when the panel length is not less than eighteen feet is

$$C = (2\,n' - 1)\,(n - n')\,\frac{E_1}{n}\,\tan\,\theta'$$

where n' (not less than $\frac{n}{2}$) is the number at that end of the panel considered nearest the middle of the span.

When the panel length is less than eighteen feet the formula is

$$C = 2\,(n' - 1)\,(n - n')\,\frac{E_1}{n}\,\tan\,\theta'$$

When there is an odd number of panels the stress in the middle panel is the same as that in the next consecutive panel.

The formula for the convex side chord stress due to E_1, when the panel length is not less than eighteen feet is

$$T = (2\,n' - 1)\,(n - n')\,\frac{E_1}{n}\,\tan\,\theta'$$

where n' (greater than $\frac{n}{2}$) is the number at that end of the panel considered *furthest* from the middle of the span. For a middle panel, when the number of panels is odd, the stress is the same as for the next consecutive panel.

If the panel length be less than eighteen feet, the formula is

$$T = 2\,(n' - 1)\,(n - n')\,\frac{E_1}{n}\,\tan\,\theta'$$

This being inapplicable to the case of a middle panel, when the number of panels is odd, the following formula is to be there used

$$T = \left(\frac{(n-1)^2}{2} - 1\right)\frac{E_1}{n}\,\tan\,\theta'$$

As before stated these stresses due to curvature can be found for all practical cases by consulting Table V.

To recapitulate: the following are the steps to be taken in calculating the stresses in the trusses by means of the tables.

1°. Make table of data.

2°. Find live and dead load stresses.

3°. Find curvature stresses upon one chord, if any.

4°. Find wind stresses on windward bottom chord with empty bridge.

5°. Find reduced dead load and the corresponding lower chord stresses, when bridge is empty.

6°. Determine whether the bottom chord will require to be stiffened; and, if so, what stresses it will have to resist.

7°. Find wind stresses on windward top chord of deck bridge, or on leeward bottom chord of through bridge, when the span is covered by the moving load.

8°. Find diminution of stresses on windward top chord of deck bridge, or increment of stresses on leeward bottom chord of through bridge due to load transferred by a wind pressure of thirty pounds per square foot on trusses and train.

9°. Combine the stresses found by 2° and 3°.

10°. Combine the stresses found by 2°, 3°, 7° and 8°.

CHAPTER IX.

STRESSES IN LATERAL SYSTEMS AND SWAY BRACING.

Without making any material error the wind pressure, for the purpose of simplifying calculation, may be considered as equally distributed between the two sides of the bridge, although the windward side does receive the larger share.

When the bridge is empty, the stress in any lateral rod can be found by the formula

$$T = \frac{n'(n'+1)}{2} \cdot \frac{W \sec \theta}{n} . \ [Eq.\ 1],$$

and that in any strut except those at the ends of the lower lateral system by the formula

$$C = \frac{n'(n'-1)+n}{2n} W. \ [Eq.\ 2.],\ *$$

where W is the sum of the pressures at a windward and leeward panel point, n the number of panels in the wind bracing counting in the two lacking at the ends of the upper lateral bracing in through bridges, n' (not less than $\frac{n}{2}$) the number at the leeward end of the rod or at either end of the strut, the panel points being marked as directed in the last chapter, and θ the angle that the rods make with the struts.

When the moving load is upon the bridge the stress in any lower lateral rod of a through or pony truss span or in any upper lateral rod of a deck span can be found by the formula

$$T = \frac{n'(n'+1)}{2} \quad \frac{W \sec \theta}{n} + \left(n' - \frac{n-1}{2}\right) W_1 \sec \theta. \ [Eq.\ 3]$$

and that in any corresponding lateral strut, except those at the ends of the lower lateral system and any strut at the middle of the span, by the formula

$$C = \frac{n'(n'-1)+n}{2n} W + \left(n' - \frac{n}{2}\right) W_1. \ [Eq.\ 4]$$

---

* This formula is not exact, and, on account of the ambiguity in the division of wind load between windward and leeward panel points, it would be difficult to make it so. It gives in every case a stress greater than that which could ever come upon the strut. The error is never very large, is greatest for the light struts near the centre of the span, and reduces to nearly zero for the heavier struts near the end of the span. The same remark applies to Eq. 4 of this chapter.

For a middle strut in a span with an even number of panels the formula is

$$C = \frac{n+2}{8} W + \tfrac{1}{2} W_1 \quad [Eq.\ 4\ A.]$$

where W is the pressure on the train which is concentrated at one panel point or 240 times the panel length in feet divided by 2000, W_1 is the panel pressure upon the lower half of the trusses and the floor system, calculated for a wind pressure of thirty pounds per square foot, and the other quantities have the same values as before.

When the span is empty the stress in the lower lateral strut at the free end of a span is given by the following formula

$$C_n = \frac{2n-1}{4} W_a + \frac{n-1}{4} W_b - \frac{l}{16} (G - 2G_1). \quad [Eq.\ 5]$$

where W_a and W_b are the panel wind loads (both windward and leeward) for the lower and upper systems respectively, l is the length of span, G the dead load per lineal foot of bridge and G_1 the released load per lineal foot under the assumed maximum wind pressure.

When the span is covered by the moving load the stress in the same member is given by

$$C_n' = \frac{2n-1}{4} W_a' + \frac{n-1}{4} W_b' - \frac{l}{16} (G' - 2\ G_1'). \quad [Eq.\ 6]$$

where W_a' is the total panel load (both windward and leeward) for the lower system i. e. the sum of the fixed and moving panel loads, W_b' the panel load (both windward and leeward) for the upper system at thirty pounds pressure per square foot, l the length of span, G' the sum of the assumed live and dead loads per lineal foot (the engine excess is not considered) and G_1' the released load per lineal foot under a pressure of thirty pounds per square foot on trusses and train.

The stresses in Eqs. 1 and 8 are to be increased for initial tension as directed in chapter VI, or, what amounts to the same thing, these stresses are to be used and the sizes of the rods determined by Table VI: and the stresses in Eqs. 2, 4, 5 and 6 are to be increased by the sum of the components in the direction of the strut of the initial tensions in all the lateral and vibration rods meeting at the windward end of the strut.

If the track upon the bridge be curved, the stresses upon that lateral system, which resists the centrifugal force, are to be found by adding to the value of W in Eqs. 8. and 4. the value of W_7 from the last chapter, and by combining these stresses with those due to the centrifugal force of the engine excess as calculated by the following formulae

$$\left.\begin{aligned} T &= 2\,(n'-1)\ \frac{E_1}{n} \sec\theta \\ C &= 2\,(n'-1)\ \frac{E_1}{n} \quad * \\ \text{and} \quad C_n &= 2\,(n-2)\ \frac{E_1}{n} \end{aligned}\right\}$$

$$\left.\begin{aligned} \text{or} \quad T &= (2\,n'-1)\ \frac{E_1}{n}\ \sec\theta \\ C &= (2\,n'-1)\ \frac{E_1}{n}\ * \\ \text{and} \quad C_n &= (2\,n-3)\ \frac{E_1}{n} \end{aligned}\right\}$$

where n' (not less than $\frac{n-1}{2}$) is the number at the leeward end of the member considered, and E_1 has the same value as in the last chapter. The first group of equations is to be used when the panel length is less than eighteen feet, and the second group for all other cases. As before C_n represents the stress in the lower lateral strut at the free end of the span. These stresses for all practical cases are given in Table V.

The method of calculating the stresses in the vertical sway bracing is as follows: the part relating to single track bridges is essentially the same as given by Prof. Wm H. Burr in his treatise on "Stresses in Bridge and Roof Trusses" The loads assumed, unless otherwise stated, are those obtained under a maximum wind pressure upon the empty bridge.

In Fig. 1 let P be the pressure concentrated at the upper panel point on one side of the bridge: it is that which comes upon a panel length of top chord, one half the area of the diagonals meeting at the panel point when the truss is single intersection or one fourth of said area when the truss is double intersection, and the portion of the post above the plane A B. Let P′ be the pressure concentrated at one end of the intermediate strut J K: it is that which comes upon one half of the post or the portion between the planes A B and C D, the latter being half way between J K and E F; also, in double intersection bridges, that which comes upon one half the area of the main diagonals and counters coupled at K, which point would then be midway between H and F.

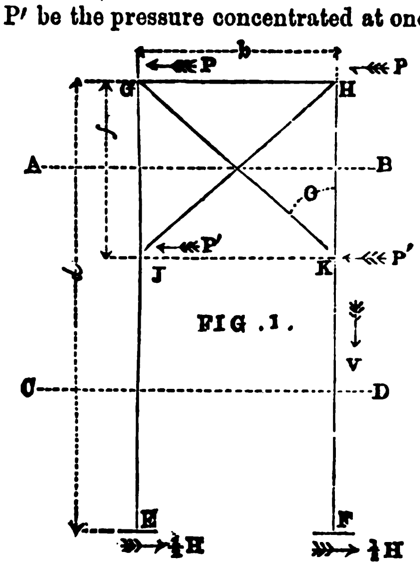


FIG. 1.

The pressures concentrated at the feet of the posts do not affect the vertical sway bracing, so are not considered.

Let d, f, b and θ reprksent the measurements indicated in Eig. 1

The total pressure $2\,(P+P')=H$ is assumed to be equally resisted by the feet of the posts: this assumption is probably as correct as any other that can be made.

Taking the centre of moments at E, the moment of the pressure is

$$2\,P\,d\ +\ 2\,P'\,(d-f)$$

* These stresses are figured as if the loads E_1 were concentrated on the chord nearest the concave side of the track. This assumption was made so as to provide for the worst possible distribution, because the exact method of division is unknown.

which can be resisted only by the moment of a released weight V upon the foot F, thus $\quad 2\,P\,d + 2\,P'\,(d-f) = Vb$

and $\quad V = \dfrac{2\,d\,(P + P') - 2\,P'f}{b}$

This released weight must pass up the vibration rod KG, causing a tension therein, represented by

$$V \sec\theta = \frac{2\,d\,(P + P') - 2\,P'f}{b} . \sec\theta ,$$

which stress must be increased for initial tension.

To find the stress on the strut JK pass a plane through the bracing cutting GH, GK and JK (HJ not being strained), take the centre of moments at G, and consider the forces acting on the leeward side of the truss, then the moment of the stress in JK will balance the moments of P′ and ½ H, thus

$$(JK) = \frac{\frac{1}{2}\,Hd - P'f}{f} = \frac{d}{f}\,(P+P') - P'$$

to which must be added the horizontal component of the initial tension in JH. (JK) represents the stress in JK.

The stress in GH is found by considering it a part of the upper lateral system and not to belong to the vertical sway bracing. But if it be supposed to belong to the latter, its stress may be found by passing a plane as before, taking the centre of moments at K, and considering only the forces acting at the leeward side of the truss, so that the moment of the stress in GH will balance the moments of the horizontal reaction at E and the pressure at G, the moment of the increase of weight at E balancing the moment of the increase of vertical reaction at that point; thus

$$GH = \frac{\frac{1}{2}\,H\,(d-f) + Pf}{f} = \frac{d}{f}\,(P + P') - P'$$

or equal to the stress in JK.

The bending moment on the post, if the lower end be considered free, would be

$$\tfrac{1}{2}\,H\,(d-f) = (P + P')\,(d-f)$$

but as the foot of the post is rigidly attached to the floor beam, it may be considered as fixed. Comparing these two conditions of the portion of the post between the foot and the intermediate strut, we see that they are similar to those of one half of a beam loaded at the middle, first with the ends supported and second with the ends fixed. It is well known that the bending moment in the latter case is only one half of that in the former, so we may conclude that the bending moment to be provided for is

$$\tfrac{1}{2}\,(P + P')\,(d-f)$$

If m be the distance between centres of gravity of post channels, the stress on *one* channel produced by the bending will be

$$C = \frac{(P+P')\,(d-f)}{2\,m}$$

The released weight V on the windward post passes down the leeward post, producing a stress equal to $\frac{V}{2}$ on each channel and making the total wind stress on one channel

$$C + \frac{V}{2}$$

It is only for light posts that this stress need be considered, because when it is to be combined with the live and dead load stresses both C and V must be calculated for a pressure of only thirty pounds per square foot. The method of providing for this wind stress is clearly indicated in Chapter VI.

All the formulae for vertical sway bracing, except that for the stress in GH, may be made applicable to the portal bracing by putting for d the length of the batter brace, for f the perpendicular distance between centre lines of upper and lower portal struts, for P' the pressure on one half of the batter brace and for P one fourth of the sum of all the pressures concentrated at windward and leeward panel points of the upper lateral system.

If P_e be the pressure concentrated at the leeward hip, the stress on the upper portal strut will be given by the formula

$$C = \frac{d}{f}(P + P') - P' + P - P_e$$

It must not be forgotten that the stresses on all portal vibration rods must be increased for initial tension or the rods be proportioned by using Table VI, and that the stress on each portal strut is to be increased by the sum of the components in the direction of the length of the strut of the initial tensions in all the rods meeting at one of its ends.

In the case of a deck bridge with a curved track thereon, the centrifugal force of the greatest panel load will affect the vertical sway bracing in the same manner as does the wind pressure; so for P must be put $P + \frac{1}{4}(W_7 + E_1)$, the last two quantities having the same signification as in Chapter VIII.

The portal bracing will also be affected by the centrifugal force of the whole train, so for P must be put one half of the greatest reaction at one end of the span, due to the combined wind pressure affecting the upper lateral system, and the centrifugal force of the whole train, or

$$\frac{n-1}{4}(W_3 + W_7) + \frac{2n-3}{2n}E_1,$$

and for P_e one half of a panel loading of wind pressure, centrifugal force of car load and centrifugal force of engine excess or $\frac{1}{2}(W_3 + W_7 + E_1)$

When there is no vertical sway-bracing, stiffness is obtained by the use of knee braces or brackets AB, CD, Fig 2, making angles of forty-five degrees with the vertical.

Let the notation be as shown in Fig. 2., V being as before the relief of weight at F. P in this case is the sum of the pressures at H and G. Taking the centre of moments at E gives.

$$Vb = Pd \text{ and } V = \frac{Pd}{b}$$

Again taking the centre of moments at A gives the value of the bending moment M on the strut at that point, thus

$$M = V(b-s) - \tfrac{1}{2}Pd = \frac{Pd}{2b}(b-2s)$$

Let h = the distance between centres of gravity of the two channels of which the upper lateral strut is composed, then the bending stress will be

$$C = \frac{M}{h} = \frac{Pd}{2bh}(b-2s)$$

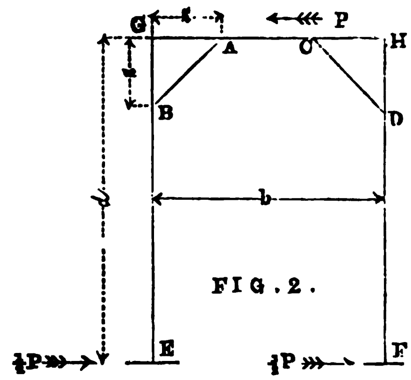


FIG. 2.

The intensity of the working bending stress being six tons, the number of square inches to be added to the area of *each* channel in order to resist the bending will be

$$A = \frac{C}{6} = \frac{Pd}{12\,bh}(b-2s)$$

The stress in A B is found by taking the centre of moments at G, and making the moment of its stress R equal to the moment of the horizontal reaction at E, thus

$$R\,s\,\sqrt{\tfrac{1}{2}} = \tfrac{1}{2}Pd$$

$$\text{and } R = \frac{Pd}{s}\sqrt{\tfrac{1}{2}} = 0.707\,\frac{Pd}{s}$$

The bending moment to be provided for in the post, considering it fixed at its foot will be

$$\frac{P}{4}(d-s)$$

and the corresponding stress on *one* channel will be given by the equation

$$C = \frac{P(d-s)}{4m}$$

where m has the same value as in the last case.

As before to make these formulae applicable to a portal, make d equal to the length of the batter brace and P equal to one half of the sum of the pressures concentrated at all the panel points of the upper lateral system.

When only one track of a double track bridge is covered by the moving load, according to the law of the lever, one truss receives more load than the other. Now if the two trusses could act independently this distribution would hold while the load covered the track; but if the two trusses were connected by perfectly inelastic vertical sway bracing they would have to deflect equally, which could only occur when the loads on each truss were equal, so that a portion of the load equal to the difference between the greater division by the law of the lever and one half of the whole load would have to be transferred by the vertical sway bracing. In reality neither of these conditions will exist, the true condition lying between the two; for the trusses do not act independently as if there were no vertical sway bracing, and the latter is far from being perfectly inelastic.

What the actual transferred load is it is impossible to say, but it will be making an error on the side of safety if it be assumed that the load is equally divided between the trusses; any extra iron that may be thereby used in the vertical sway bracing will be well employed, for it will be in a good place to resist vibration.

Under this assumption let us investigate the stresses in the bracing. Let the notation be as in Fig. 3, R and R' being the reactions due to the weight W, distributed according to the law of the lever, so that

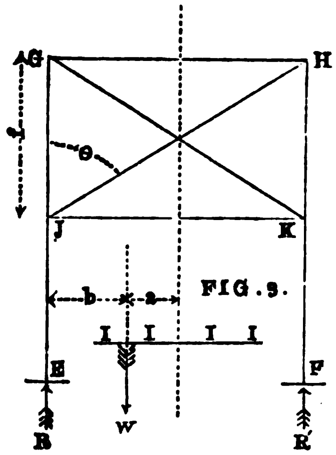


FIG. 3.

$$R = W\frac{2a+b}{2(a+b)}$$

Let G be the weight transferred by the bracing, then

$$G = R - \tfrac{1}{2}W = W\left(\frac{2a+b}{2(a+b)} - \tfrac{1}{2}\right) = \frac{Wa}{2(a+b)}$$

The stress in the vibration rod is therefore

$$T = G \sec\theta = \frac{Wa \sec\theta}{2(a+b)}$$

The stress in J K is found by passing a plane to cut G H, J K and J H, supposing that the only weight acting is $\frac{Wa}{2(a+b)}$ at E, and taking the centre of moments at H. This gives

$$(JK) = \frac{Wa}{2(a+b)} \cdot \frac{2(a+b)}{f} = \frac{Wa}{f}$$

Again taking the centre of moments at J and using the same cutting plane we find the stress in G H to be zero; for the moment of the increase of weight at F is balanced by the moment of the increase of reaction at that point, making the resultant moment of the external forces zero, and the stresses in J H and J K, having no lever arms, their moments are zero, consequently the moment of the stress in G H is zero and the stress itself zero.

To find the bending effect upon the post at K let us pass a plane cutting K F and J E, and take the centre of moments at K, then

$$M = \frac{aW}{2(a+b)}\left[2(a+b)\right] = Wa$$

If h be the distance between centres of gravity of post channels and an intensity of four tons be employed the area of *one* channel necessary to resist this bending will be

$$A = \frac{M}{4h} = \frac{Wa}{4h}$$

But as this effect does not exist at the same time as the maximum load stress upon the post H F, it need be considered only when the post is very light.

To ascertain whether it needs consideration, find the stress on the post with one train only upon the bridge, reaching from the most remote end of the span to the foot of the post considered, and under the supposition of an equal distribution of the train load between the trusses; then proportion the post to resist this stress according to the method to be explained in Chapter XI, and to one half the section thus found add the value of A in the last equation. If the sum exceed the area of one of the post channels required to resist the maximum live and dead load stresses when both tracks are partially covered by the assumed moving loads, then the post section is to be increased accordingly.

The vibration rods should be proportioned to resist the transferred load stress using an intensity of five tons, or to resist the sum of the transferred load stress and the wind stress under thirty pounds pressure, using an intensity of seven and a half tons. If the former give the greater section, then the strut should be proportioned to resist the transferred load stress using the intensity given in Table VIII, but, if not, it should be proportioned to resist the sum of the transferred load stress and the wind stress under thirty pounds pressure using the intensity given in Table IX. It must not be forgotton that the effect of initial tension is to be allowed for in proportioning both vibration rods and intermediate struts.

In double track bridges without vertical sway bracing the trusses will probably act nearly independently, but of this one cannot be certain, so it may be well to calculate the formula for the bending effect on the upper lateral struts due to the transferred load under the assumption of equal distribution between trusses, and apply it to a practical case.

Let the notation be the same as in Fig. 3., but let s have the same signification as in Fig. 2., then the bending moment upon the strut will be

$$M = \frac{Wa}{2(a+b)}[2(a+b) - s]$$

If the distance between centres of gravity of strut channels be h and the intensity of working compressive stress be six tons, the area required for one channel to resist bending will be

$$A = \frac{M}{6h} = \frac{Wa\ [2(a+b) - s]}{12h\ (a+b)}$$

Let us take the case of a 20′ panel and assume $a + b = 12$, $h = 1$, and $s = 7$, we will then have the following data

$$\begin{aligned} W &= 21.2 \\ a &= 4.8 \\ b &= 7.2 \\ h &= 1.0 \\ \text{and } s &= 7.0 \end{aligned}$$

which substituted above gives

$$A = \frac{21.2 \times 4.8 \times 17}{12 \times 12} = 12 \quad \text{square inches,}$$

corresponding to a 40 pound channel, which is far greater than would ever be used in practice. Hence we may conclude that it will be necessary to assume that the trusses deflect independently in all cases where their depth is not great enough to permit of the use of vertical sway bracing.

It might be well however to increase the sectional area of the upper lateral struts in order to resist the racking effect which the vibration of passing loads produces in these members. It is difficult to say what the amount of increase should be, but the author considers that if six inch channels weighing eight or ten pounds per lineal foot and spaced ten or twelve inches apart be employed, the struts will be strong enough.

It is obvious that no unequal distribution of moving load can affect the portal bracing, for the loads at the feet of the batter braces rest directly upon the foundations, and those at the first panel points produce an inconsiderable shortening of the batter braces.

CHAPTER X.

RIVETTING.

The subject of rivetting is one which seldom, if ever, receives its due amount of attention from bridge designers.

Many structures otherwise very strong are extremely weak in detail, owing to the insufficient number of rivets employed in the connections, and to their improper arrangement.

The principal rules for rivetting have been given in Chapter VI.

Rivets should be proportioned for bending and bearing pressure, i. e. for any given connection the number of rivets necessary to resist properly each of these stresses should be determined, and the greater number chosen.

Table XVII gives the working bending moments and bearing pressures for rivets in trusses and floor systems. It can be used for lateral systems also by making a proper reduction in the calculated stress: this will be exemplified in Chapter XVIII.

In this table the first two horizontal lines containing vulgar fractions and decimals give the widths of bearings, and the other horizontal lines in the portion relating to bearing give the working bearing stresses for rivets of different diameters. The rest of the table needs no explanation.

The diameters for rivets in railroad bridges vary from half an inch to an inch, but by far the larger number are from three quarters to seven eighths.

When two plates are rivetted together, the rivets, being driven when hot, contract or tend to contract in length when cooled, thus drawing the plates together and producing a friction which it is necessary to overcome before shear can come upon the rivets. Whether this friction will continue indefinitely is doubtful, for rivets occasionally become loosened when the structure is subjected to oft repeated loads; so it is not legitimate to depend upon the friction in order to reduce the number of rivets. Perhaps it is on account of this factor that rivets are seldom, if ever, proportioned to resist the bending moments that come upon them, notwithstanding the fact that it is this last consideration which in most cases should determine the number to be employed.

It will probably have been noticed by the reader that shearing stress upon rivets has been entirely omitted from consideration. The author would hesitate before making the broad assertion that rivets cannot shear, although it is probable that bending is the stress which ruptures rivets that are generally considered sdeareh:

this much though he will state as the result of both theoretical investigation and many practical cases of designing, *that when rivets are proportioned for bending they will have more than sufficient strength to resist shear.* Sharp edges on rivet holes will certainly cut the rivets, but this is not shear proper; and it may be possible that there is a certain kind of fixedness about a well driven rivet, which will make the bending moment less than its calculated value, but this fixedness is obtained by a high initial tension, which increases the stress on the tension side of the rivet subjected to bending. As this initial tension cannot be measured or calculated, it is safer to assume no reduction of bending moment on account of its existence.

Countersinking is a term used to denote the sinking of rivet heads into the plate so as to make them flush with its surface. The least allowable depth for the countersinking is a quarter of an inch, and the least thickness of plate used for this purpose should be three eighths of an inch: for rivets over three quarters of an inch in diameter these dimensions should be increased by an eighth of an inch. Rivets may be countersunk at one or both ends.

Making parallel rows of rivets staggered avoids unnecessary weakening of the parts rivetted together.

There has been much discussion as to whether punched or drilled holes are preferable, the general conclusion being that drilled holes weaken the plates less, and, when slightly countersunk do not increase the shear upon the rivets; but that punched holes are so much more economical as regards shopwork that, when properly made, they are preferable to drilled ones. The improvements made of late years in rivetting machines have increased the efficiency of work with punched rivet holes.

Should for any reason it ever be necessary in bridge designing to put a rivet through a plate whose thickness is greater than the diameter of the rivet, the rivet hole should be drilled. Machine rivetting is preferable to hand rivetting, but there are cases when the latter has to be employed. Field rivetting is always inferior to shop rivetting, so should be avoided as much as possible in making designs. When a stress is transmitted from one plate through one or more plates to another plate, the number of rivets must be increased. The rule given by Dr. Weyrauch is that "for every single shear connection the indirect force transferrence requires for m intermediate plates $m+1$ times as many rivets as for direct transferrence".
Keeping this in view the designer will avoid using more than one flange plate in floor beams or more than one plate for covering the channels of top chords.

CHAPTER XI.

PROPORTIONING OF MAIN MEMBERS OF TRUSSES, LATERAL SYSTEMS, AND SWAY BRACING.

Having found all the stresses in the main members of the truss and in those of the lateral systems and sway bracing, and having written them alongside the respective members in the diagrams, the next step is to calculate the sections required. The diagrams for the lateral systems and sway bracing may be roughly drawn in pencil; for they need not be preserved, as the sizes of the members are to be written on the truss diagram.

For the tension members of the trusses, the sections required can be found by dividing the stresses on the diagram by the proper intensities of working-stress, as given in Chapter VI, remembering that the intensities for main diagonals are to be interpolated. When found, the required areas for the sections should be written on the diagram, after the stresses, prefixing them with the letters S. R. (section required), as shown on plate XV. It must not be forgotten that the sections required for the bottom chords must be calculated for live and dead load stresses with an intensity of five tons and for combined live load, dead load and wind stresses with an intensity of seven and a half tons. Then, by using the proper tables in Chapter II, there can be found the sizes of tension members which will give *at least* the section required.

The stresses in the counters are to be increased for initial tension by the amounts given in Chapter VI, or what amounts to the same thing, the size required can be found from Table VI. by looking down the column headed "Intensity of Working Stress = 4 tons" until a stress is reached which is equal to or greater than one-half or the whole of the stress on the diagram, according to whether double or single counters be employed; then, by following the horizontal line which contains this stress, either to right or left, will be found the size of the counters or counter required.

The sizes of hip verticals can be found without calculation from Table VII.

In pony trusses having less than five panels it is necessary to make them stiffened, using either trussed bars or two channels laced or latticed.

As stated in Chapter VI, if the former be employed the intensity of working stress is to be reduced to three tons, and, if the latter, the effective area of the webs alone is to be relied upon to resist tension.

The sizes of the lateral and vibration rods can be found from the last mentioned table by looking in the column headed "Intensity of Working Stress = 7.5 tons" in the same manner as explained for counters.

If the panel length correspond with the one given in Table I, or if it do not differ greatly therefrom, there need be no calculations made for stresses in the lateral systems and sway bracing; because the dimensions of all the struts and rods for these systems are given in Table XIII. In that table the dimensions in the column marked "Pan. 1" are the sections respectively of the portal vibration rods (if any), the lower portal struts (if any), the end lower lateral rods, and the lower lateral strut at the free end.

Those in the second column are the sections respectively of the upper portal struts, the upper lateral rods, the lower lateral rods of the second panel, and the lower lateral strut at the first panel point. Those in the other columns are respectively the sections of the upper lateral strut, the upper lateral rods, the vibration rods (if any), the intermediate strut (if any) the lower lateral rods and and the lower lateral strut. Thus the portal rods, lower portal struts, end lower lateral rods, and end lower lateral strut are assumed to belong to the first panel; the upper portal strut, end upper lateral rods, second panel lower lateral rods and the lower lateral strut at the first panel point to belong to the second panel; the end upper lateral strut, the vibration rods and intermediate strut attached to the first pair of posts, the lower lateral rods of the third panel and the lower lateral strut at the second panel point to belong to the third panel etc. etc. Spans above one hundred and fifty feet in length have vertical sway bracing.

If the counter stresses be large, it is preferable to use double counters: sometimes both single and double counters are employed in the same truss. Where there is an odd number of panels, the centre diagonals should be made double and adjustable. The number of main diagonals per panel is generally two; but, if the sections become so great as to necessitate excessively large chord pins, it is better to employ four; placing two inside, and two outside, of the top chord and posts. The widths of the main diagonals should, for the sake of appearance, increase from the centre of the bridge to the ends. For the same reason, it is well to have all the chord bars of the same, or nearly the same, depth; the correct area of section being obtained for each panel by varying the thickness and the number per panel. In large bridges it is permissible to reduce the depth of the chord bars towards the ends of the span in order to economize on the pins. It is also permissible, when there are several chord bars in the same panel, to employ depths varying by a quarter of an inch, provided that the bars of smaller depth be placed on the inside.

If the bottom chord contain a channel strut, it will be necessary to proportion this member before determining the number and sizes of the bars, which is accomplished by subtracting from the total section required the *effctive area* of the *webs* of the channels, and using the remainder as the section required for the bars.

In order that the strut may never be subjected to more than the stress assigned to it, each pin hole should be elongated towards the nearer end of the span a certain amount which can be determined by the following method.

Let A = the effective area of the strut webs in the middle panel, or panel

nearest the centre of the span.

B = the total area of the chord bars in the same panel.

C = the gross sectional area of the chord strut.

T = the intensity of working tensile stress for the chord bars.

and T' = the intensity of tensile stress upon the gross section of the strut when the bars are subjected to T, and when the strut is pulling the proper amount.

then

$$A\,T = C\,T' \text{ or } T' = \frac{A\,T}{C}$$

If E be the coefficient of elasticity of the iron the stretch of each chord bar will be

$$S = \frac{T\,l}{E}$$

where l is the panel length

The stretch of the strut due to the stress A T will be

$$S' = \frac{T'\,l}{E} = \frac{A\,T\,l}{C\mathrm{E}}$$

Now, if the number of panels in the span be even, the elongation of the pin hole at the $(\frac{n}{2} + 1)^{\text{th}}$ panel point will be

$$S - S' = \frac{T\,l}{E}\left(1 - \frac{A}{C}\right)$$

But if the number of panels be odd the elongation of each pin hole nearest the centre of the span would be

$$\tfrac{1}{2}\,(S - S') = \frac{T\,l}{2\,E}\left(1 - \frac{A}{C}\right)$$

The stretch of the chord bars in the next panel towards the end of the span will be as before

$$S = \frac{T\,l}{E}$$

and that of the strut

$$S'' = \frac{T''l}{E} = \frac{A'\,T\,l}{C'\,E}$$

Now if, as should generally be the case, the strut has the same sectional area from end to end of span, A = A$'$, C = C$'$ and therefore $S'' = S'$

For convenience of demonstration let it be assumed that the number of panels in the span is even, then the total stretch of the chord bars in the two panels lying to one side of the middle of the span will be 2 S, and the stretch of the strut in the same two panels will be 2 S$'$; therefore the elongation of the second pin hole from the middle should be 2 $(S - S')$

$$= \frac{2\,T\,l}{E}\left(1 - \frac{A}{C}\right)$$

Similarly that of the next pin hole would be

$$\frac{3\,T\,l}{E}\left(1 - \frac{A}{C}\right)$$

Finally, if n and n' have the usual signification the elongation of the pin hole at

the $(n')^{th}$ panel point should be

$$\left(n' - \frac{n}{2}\right)\frac{Tl}{E}\left(1 - \frac{A}{C}\right)$$

The following important fact should never be forgotten in designing this member —" if the pins pass through the holes in the struts and lie as nearly as possible to the centre of the span, the distances between their centres should be l plus the allowable play of the pin in the hole of an eye-bar or $l + \frac{1}{50}''$."

Let us anticipate a little and consider the case of the bottom chord strut designed in chapter XVIII: its gross sectional area is 5. 7 square inches and its effective area 2. 8 square inches, so that $\frac{A}{C}$ may be take equal to $\frac{1}{2}$.

Let us assume $l = 22$ feet, and $E = 28,000,000$ pounds or 14,000 tons.

Then the elongation at the $(n')^{th}$ panel point is

$$\left(n' - \frac{n}{2}\right)\frac{5 \times 22 \times 12}{14,000} \times \frac{1}{2} = 0.047\left(n' - \frac{n}{2}\right)$$

With an even number of panels the elongations for the various pin holes, beginning at the one next to but not at the middle of the span would be

0.047 ″, 0.094″, 0.141 ″, 0.188 ″, 0.235 ″, 0.282″ &c.

And with an odd number of panels they would be

0.024 ″, 0.071 ″, 0.118 ″, 0.165 ″, 0.212 ″, 0.259, 0.306 ″ &c.

But in connection with this investigation there is another point which must be considered; viz. that when the bridge is empty the dead load should generally be great enough to put the chord strut in tension near the ends of the span, in order to prevent both vibration and undue stresses on the chord bars at these places. Whether this condition exist can be determined by finding the dead load stress at the middle of the span, dividing it by five and comparing the ratio, which this quantity bears to the *actual sections of the chord bars* at this place, with the ratio $\frac{A}{C}$. If the former be the greater the strut will be in tension, and all will be right. Otherwise it may be necessary to reduce the amount of the elongation of those two or three pin holes nearest the ends of the span; because, supposing that the strut were not in tension when the bridge is empty, as soon as an engine covers one or two panels at the end of the span, the chord bars in these panels will be subjected to a greater intensity of stress than will those in the other panels, and if there be any play in the strut eyes which is not already taken up by the dead load, these chord bars may be stretched more than the allowable amount $\frac{T}{E}$, even before the strut comes into action as a tension member; but, if the play be taken up by the dead load, and the chord panels near one end of the span be strained more than the others, the strut will immediately begin to do its share of the work as soon as the live load is applied, and none of the chord bars will be overstrained.

The danger of overstraining the chord bars of the end panels necessarily increases with the ratio $\frac{A}{B}$ or that of $\frac{C}{B}$. These ratios increase with the length of span, but fortunately the ratio of dead load to total load also increases, causing the danger of overstraining to diminish.

For example in the 300′ span on plate XLII the dead load stress at the middle panel of the bottom chord is about 141 tons, which divided by five gives 28. 2 square

inches. The ratio of the latter to the total section of the chord bars is $\frac{28.3}{52} = 0.54$

The ratio of $\frac{A}{C}$ is $\frac{8}{13.8} = 0.58$, showing that if the elongations of the strut eyes be made according to the preceding theory, there will be a little play not taken up by the dead load, so it will be necessary to make the elongations at say the shoes and first panel points equal to that at the second panel points.

Again in the 200′ span on Plate XXXII the dead load stress at the middle is about 58 tons making the section for same 11.6 square inches and the ratio $\frac{11.6}{23.16} = 0.5$. The ratio of $\frac{A}{C}$ is $\frac{5.4}{9.6} = 0.56$, or greater than that last found, showing that in this case also the elongations of the pin holes at the shoes and first panel points must be reduced from the theoretical amounts.

Finally in the 120′ span on Plate XXII the dead load stress at the middle is about 23.28 tons, and the ratio of sections $\frac{4.64}{11.2} = 0.414$; while the ratio of $\frac{A}{C}$ is $\frac{2.8}{5.1} = 0,55$. This great difference is not of much importance for the ratio of $\frac{A}{B}$ in the end panels is so small that the chord bars alone are about sufficient to take up the stresses caused by an engine load or engine loads near one end of the bridge, consequently if the pins do lie loosely in the strut eyes when the bridge is empty, no harm will be done; for vibration may be avoided by careful chord packing.

Hence we may conclude that the preceding theory of strut eye elongation may be adopted in all cases at all the panel points except one or two at each end of long spans, at which places they may be made equal to that at the next panel point towards the middle of the span.

"Chord packing" is a term applied to the arrangement of the chord bars, chord strut, diagonals, posts, and beam hangers upon the bottom chord pins. It is a matter of great importance, but is very often neglected. The three principal considerations to be kept in mind while arranging the packing are, that the bending moments on the pins are to be made as small as possible, that the packing is to be made as close as circumstances will permit, and that there be sufficient clearance to avoid all chance of finding the space between the post channels too narrow when the bridge is being erected.

The width of the packing is dependent, not only upon the number and thickness of the bars, but also upon the width of the top chord plate. The latter is often, in its turn, dependent upon the chord packing.

The best arrangement is to pack the main diagonals, counters, chord strut and beam hangers inside of the posts, and the chord bars outside; bringing the latter, however, within the batter braces at the shoes, unless the end panel contain four bars per truss, when two should go outside, and two inside. It is not absolutely necessary that the chord bars pull in the exact line of the trusses; an inch or two of deflection in twenty feet being scarcely noticeable and making no appreciable difference in the length of the bar; nevertheless it is better to make the bars as nearly as possible parallel to the planes of the trusses. The main diagonals should be placed next the post, then the beam hangers, and inside of all the counter or counters and chord strut, if there be one, any vacant space thus left being adjusted with fillers. The arrangement of the chord bars will be treated further on.

The width of the top chord or batter brace plate is dependent upon the depth of the channels, as the transverse distance between the centre lines of rivets, which attach the channels to the plate, should never be less than the depth of the channels: the chord packing often demands the use of wider plate. The least widths and thicknesses are given in Chapter VI

In pony trusses the channels are spread apart and the width of plate increased to give lateral stiffness to the truss. To proportion the top chord or batter brace for a given stress, assume the depth of the channels and divide the length of panel or batter brace by it, both dimensions being expressed in the same unit.

Referring to Table VIII, look down the column marked "Ratio of L to D", until the ratio just found is reached, the number to the right, in the first of the three columns, is the intensity of working-stress to be used. The three columns are for the three cases,—both ends fixed, one end fixed and one end hinged, and both ends hinged, marked ▨▨, ▨●, and ●● respectively. Then, to find the area of the top chord or batter brace, divide the stress given on the diagram by the intensity of working-stress taken from the table; from the quotient subtract the area of the top plate, and divide the remainder by two: the final quotient will be the area of each channel. This calculation should be made with both the stress in the panel nearest the middle of the span and that in the end one, or, in long spans, that in the one next to the end. If, then, with the depth of channel assumed, it be found that there is, in the table of channel sections employed, a light channel that will not be much too heavy for the end, and a heavier one suitable for the middle of the chord, all right: if not, another trial must be made, with a channel of a different depth. The greater the depth of channel, the less the ratio of length of strut to diameter, and consequently the greater the intensity of working-stress, and the less the sectional area required: so, generally speaking, it is well to use the lightest and deepest channels possible, unless the saving in section be small, when it will be more economical, for other reasons, to use the next smaller depth. These reasons will be given in Chapter XVI. The dimensions of the channels and plate should be written on the diagram of stresses as shown on Plate XV.

The sizes of the post channels are to be found in a similar manner to the one just described, with these two exceptions,—that the column for two hinged ends is to be used, and that there is no plate.

After all the posts are proportioned, the light ones should be tested according to the method explained in Chapter IX to see if they are strong enough to resist the combined effects of loads and wind pressure. If not the sections should be increased so as to make them strong enough.

In high double intersection bridges where the diagonals are halved, and connected by pins passing through the middle of the post channels, as shown in Plate I, the posts may be proportioned for half-length with both ends hinged; but in this case the counters must extend to the ends of the span, although there be no stress in some of them, for the purpose of preventing the posts from moving laterally at the middle.

All lateral and portal struts, also the intermediate struts in double track bridges,

are to be proportioned by using Table IX for one fixed and one hinged end, and adding, if necessary, to the section thus found enough area to resist bending as determined in Chapter IX.

These struts do not really have one end fixed and the other hinged, but the strength of a strut connected by abutting jaws is intermediate between that of a strut of the same size with both ends fixed and that of another strut of the same size with both ends hinged. This is because the inaccuracies of shop work may cause a slight deviation from axial bearing.

It is not positively necessary to use a lateral strut between pedestals at the fixed end of all spans, but it is much better to do so, not only to distribute the horizontal reactions, but to keep the chords in line; for there is necessarily a little play in the anchor bolt holes.

There is generally no objection to making these struts lighter than those at the free end of the span, but it will often be found that considerations of detail will necessitate the use of a strut nearly as large as that at the free end. For instance, if there be a vertical pin 3½″ in diameter passing through the jaw, it would entirely cut away the flanges of an I-beam, and would leave but little iron in two 4″ channels so two 5″ channels would be required.

Again, the component of the stress in the end lower lateral rod in a direction perpendicular to the strut will produce a bending on the connection, and an inequality of shear upon the rivets which connect the jaw plate to the strut. To resist this these rivets should be spread apart as much as possible, and the nut which connects the jaw to the chord pin should be screwed up very tightly and perhaps locked. It is obvious that this objectionable tendency to bending is less the nearer the vertical pin is to the end of the strut.

The total working stresses (*not intensities*) for I beams used as intermediate struts in single track bridges are given in Table X. Both ends may be considered fixed. When the I-beam strut is supposed to bend in a vertical plane, its length should be taken equal to the distance between the points of attachment of the brackets; but, when it is assumed to bend in a horizontal plane, its length must be taken equal to the distance between opposite posts of the trusses.

Brackets should extend inward and downward, from about four feet in narrow bridges, to about six feet in wide ones. The sway bracing given in Table XIII was proportioned for brackets of these dimensions. Brackets beneath intermediate struts not only serve to stiffen the struts, but add to the appearance of the bridge.

To proportion the bottom chord strut, find the greatest stress in any panel, assume the depth of the channel, which should be made so great that the bottom chord pins will not take too much material out of the webs, and divide this stress by the intensity found in Table VIII for both ends fixed. If the sectional area thus found be small, it can be increased without causing much waste, for any area added to the webs of the channels is merely taken off that of the chord bars.

If the stiffening be accomplished by trussing the inner chord bars, it must not be forgotten that the intensity of working tensile stress is to be reduced to four tons for the bars so trussed. Struts of trussed bars may be assumed to have an intensity of working compressive stress equal to one ton for short panels, and half a ton for long ones.

CHAPTER XII.

PROPORTIONING OF TRACK STRINGERS, PLATE GIRDERS, FLOOR BEAMS AND BEAM HANGERS.

The economic depth for a track stringer or plate girder will not generally vary much from one eighth of the span, although English engineers usually adopt one twelfth. The best American specifications give preference to girders in which the depth is not less than one tenth of the span. Other things being equal the deeper the girder the less will it deflect, so a depth of one eighth is preferable to one of one twelfth of the span, unless it necessitate the use of more iron, which it should not. There can be considerable variation in the designing of girders, which will involve an appreciable difference in their weights and economic depths.

As the function of the web is to resist shear alone, the formula for the area of the upper flange of a girder will be

$$A = \frac{M}{T d}$$

and that for the lower flange

$$A_1 = \frac{M}{T d} + A''$$

where M is the greatest bending moment on the girder at the point considered, $T =$ the intensity of working stress $=$ 4 tons for built girders or 5 tons for rolled ones, d the depth between centres of gravity of flanges, and A'' the area lost from the lower flange at the point considered by the rivet holes. As a table of equivalent uniformly distributed live loads for all cases is given in Chapter VI and another corrected for shock in Chapter VII, the value of the greatest bending moment at any point of a girder will be given by the equation

$$M = \frac{w x}{2} \ (l - x)$$

which for the middle point becomes

$$M_1 = \frac{w\,l^2}{8}$$

w being the total uniformly distributed live and dead load per lineal foot including the allowance for shock, l the length of the beam and x the distance from the nearer end to the point considered.

The shearing stress on the web should never exceed 2 A', where A' is the area of the section of the web: except for extremely shallow girders, which are seldom permissible in good practice, the shear can never be as great as 2 A', so this consideration may be usually omitted. For reasons stated in the last chapter, a multiplicity of parts in the flanges of beams or girders is to be avoided, so it is well to use no top plate and but one bottom plate for track stringers, and never more than three thicknesses of plate for the flange of the largest girder. The rules for web stiffening are given in Chapter VI. The sizes of stiffening angles may vary from $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ to $3'' \times 3'' \times \frac{3}{8}''$.

When the panel length does not exceed fifteen feet no bracing frames between track stringers will be needed, but for greater panel lengths there must be one frame at the middle of the stringers, and, if the latter rest on top of the floor beams, one near each end also. A bracing frame can be made out of one piece of $3'' \times 3''$ angle iron not less than $\frac{3}{8}''$ thick, by cutting right angled nicks out of one leg, bending the iron until the cut edges meet and welding the junction.

For plate girders consecutive bracing frames should not be separated by more than seven feet, and those at the ends should be made stronger than the others. As stated in Chapter VI, when the length of span exceeds twenty-five feet, the intermediate frames are to be replaced by diagonal braces of angle iron both above and below, the braces making with each other angles of about sixty degrees.

The only essential difference between a plate girder and a track stringer is in the inclined stiffeners at the ends; and those track stringers which rest upon the masonry should be provided with this detail.

The following table gives with sufficient exactitude for all practical purposes the total uniformly distributed load per lineal foot for *one* track stringer or plate girder of any length: it includes the live load with the allowance for shock, the constant track load and the weight of the girder.

Span	Unif. Load.	Span	Unif. Load.
0 to 18′	2870 pounds	33′ and 34′	2370 pounds
19′ and 20′	2810 „	35′ „ 36′	2300 „
21′ „ 22′	2750 „	37′ „ 38′	2230 „
23′ „ 24′	2690 „	39′, 40′ and 41′	2160 „
25′ „ 26′	2630 „	42′, 43′ „ 44′	2090 „
27′ „ 28′	2570 „	45′, 46′ „ 47′	2020 „
29′ „ 30′	2510 „	48′, 49′ „ 50′	1950 „
31′ „ 32′	2440 „	51′ to 60′	1880 „

The value of the greatest bending moment for a floor beam for a single track bridge may be determined as follows.

Let W = the load applied at the bearing of two abutting track stringers : it includes one half of the greatest combined engine and car loads that can be concentrated at a panel point, the allowance for shock, one half the constant track load per lineal foot multiplied by the panel length, and the total weight of one stringer.

w = the weight of floor beam per lineal foot

b = the perpendicular distance between central planes of trusses.

x = the distance from the central plane of the nearer truss to the point of the beam considered ;

then for any point between the stringers the moment is given almost exactly by the equation

$$M = W\left(\frac{b}{2} - 1.85\right) + \frac{1}{8}\, w\, b^2$$

and for any exterior point by the equation

$$M = W x + \frac{w x}{2}\,(b - x)$$

For a floor beam of a double track bridge the moment at any point between the inner track rails is given approximately by the equation

$$M = W\,(\,b - 9.88\,) + \frac{1}{8}\, w\, b^2$$

Under the outer rails it will be given approximately by the equation

$$M = W\,(\,b - 13.58\,) + \frac{1}{12}\, w\, b^2\,)$$

and for any exterior point exactly by the equation

$$M = 2\,W x + \frac{w x}{2}\,(\,b - x\,)$$

The area of a flange for a floor beam is determined by substituting the value of M in one of the equations

$$A = \frac{M}{Td} \;\; or \;\; A_1 = \frac{M}{Td} + A''$$

which were given for girders.

The economic depth for floor beams of single track bridges will vary from $\frac{b}{7}$ for short panels to $\frac{b}{6}$ for long ones : for double track bridges the corresponding ratios will probably be found to be $\frac{b}{6}$ and $\frac{b}{5}$.

The values of W for the different panel lengths are given approximately in the following table.

Pan. Length.	W	Pan. Length.	W
10'	13.84 tons	18'	19.79 tons
11'	14.85 „	19'	20.43 „
12'	15.79 „	20'	21.20 „
13'	16.67 „	21'	21.95 „
14'	17.36 „	22'	22.62 „
15'	18.06 „	23'	23.26 „
16'	18.73 „	24'	24.00 „
17'	19.28 „	25'	24.64 „

For the floor beams of single track bridges the value of W may be taken from 0.06 ton for short panels to 0.08 ton for long panels; and the corresponding values for floor beams of double track bridges at 0.12 ton and 0.15 ton.

The method of determining the rivet spacing in the flanges of plate girders, track stringers and floor beams was indicated in Chapter VI: it will be further illustrated by an example. To find the number of rivets necessary to connect a track stringer to a floor beam where the former abuts against the latter.

let W = as before the total reaction upon the floor beam caused by the total loads on two abutting stringers, including allowance for shock.
t = thickness of web of floor beam
t' = thickness of web of stringer
t'' = thickness of a connecting bent plate
p = the intensity of working bearing pressure
m = the working bending moment for one of the rivets used in the connection
and d = diameter of same rivet.

then the reaction at one end of a stringer is $\frac{W}{2}$, and the working bearing pressure for one rivet passing through the web of the stringer is $p\,t'\,d$; consequently the total number of rivets through the stringer web necessary to resist the bearing pressure will be given by the equation

$$n = \frac{W}{2\,p\,t'd}$$

The stress $\frac{W}{2}$ is equally divided between the two connecting plates, making the stress upon each equal to $\frac{W}{4}$ and its moment equal to $\frac{W}{4} \times \frac{t' + t''}{2}$, therefore the number of rivets through the stringer web necessary to resist bending will be given by the equation

$$n' = \frac{W\,(t' + t'')}{8\,m}$$

The greater of the two numbers n and n' is to be taken as the proper number of rivets to use. Additional strength is gained for this connection by the supporting shelf, but it is well not to depend thereon, as the bearing on the shelf may be imperfect, and this rivetted connection is more affected by impact than any other rivetted connection in the bridge.

The number of rivets required to attach each bent connecting plate to the floor beam as far as bending is concerned will be given by the equation

$$n'' = \frac{W\,(t + t'')}{8\,m}$$

But for bearing upon the floor beam web, the total pressure being W, the number of rivets for connecting each bent plate will be given by the equation

$$n''' = \frac{W}{2\,p\,t\,d}$$

As before the greater of the two numbers n'' and n''' is to be used.

In general $t = t' = t'' = \frac{3}{8}''$, $p = 6$ tons, and if three quarter inch rivets be employed, $d = 0.75''$, and $m = 0.811$ inch ton (vide Table XVIII).
Substituting gives

$$n = n''' = \frac{8\,W}{27} \text{ and } n' = n'' = \frac{10\,W}{88} \text{ nearly}$$

and as W varies from 13.84 tons to 24.64 tons the number of rivets passing through each leg of each bent connecting plate will vary from five to eight, which numbers it would be well to increase to seven and eleven.

In plate girder designing care must be taken to so stagger the rows of rivet holes passing through the flanges that the latter will be weakened as little as possible. If there be but one plate a single row of rivets on each side spaced about five inches will be sufficient; if there be two plates, a single row spaced three and a half inches will answer; but if there be three plates, they must be wide enough to contain two rows of rivets on a side, with a spacing of five inches. It is better to use $\frac{7}{8}''$ rivets to pass through three thicknesses of plate and the leg of a flange angle, or even to pass through two thicknesses and the leg, if the plates be as thick as half an inch.

Beam hangers may be proportioned by the equation

$$A = \frac{W_1}{24},$$

where A is the area of the section of one leg of a hanger, and W_1 the total weight of a floor beam and its load not including any allowance for shock, the latter being provided for by the low intensity of working stress.

Let us take, to illustrate the desgning of a girder, the case of a track stringer for a 21′ panel. The uniformly distributed live and dead load including shock is given in the first table of this chapter as 2750 pounds = 1.375 tons per lineal foot. The moment at the centre is

$$M = \frac{1}{8}\, w\, l^2 = \frac{1.375 \times 21 \times 21}{8} = 75.8 \text{ foot tons.}$$

Let us assume the economic depth of web to be 29″ and take the thickness as $\frac{3}{8}''$, making d about 26.5″ = 2.21 feet, therefore

$$A = \frac{75.8}{4 \times 2.21} = 8.58 \text{ square inches}$$

From the well known fact that a bar of wrought iron one square inch in section and three feet long weighs almost exactly ten pounds, we can determine the weight per foot of *each* flange angle by multiplying A by ten and dividing by six.

This gives the weight per foot to be 14.3 pounds. Consulting Carnegie's table of angle irons given in Chapter II, we find that the nearest size is a $8'' \times 3\frac{1}{2}'' \times \frac{11}{16}''$ angle, weighing 14.4 pounds per foot, which section we will adopt.

The area of the section of the bottom flange is $8.58\,\square'' + A''$, where A'' is equal to the diameter of a rivet hole multiplied by twice the thickness of the leg of one of the lower flange angles. The thickness of the upper angle legs being $\frac{11}{16}''$, we can assume that of the lower legs as $\frac{1}{2}''$, for there is to be a bottom plate. Let us use $\frac{13}{16}''$ rivets for both flanges, because of the rather large thickness of the upper flange

angles, and let us use a bottom plate $\frac{7}{16}'' \times 8''$. By a careful arrangement of the three rows of rivet holes through the legs of the angles, we can have no section weakened by more than one rivet hole, nevertheless it will be better to add say four tenths of a square inch to A'' as thus found, because the holes of the rows in the vertical and horizontal legs come pretty closely together. A″ may therefore be taken equal to $2 \times \frac{1}{2} \times \frac{7}{8} + 0.4 = 1.28$ square inches, making the area of the bottom flange $8.58 + 1.28 = 9.86$ square inches. Subtracting therefrom the area of the plate or $\frac{7}{16} \times 8 = 8.5$ square inches, leaves 6.86 square inches as the area of the two angles, which multiplied by ten sixths gives 10.6 pounds as the weight per foot of each angle. The most suitable section given in the table last used is $8\frac{1}{2}'' \times 4'' \times \frac{7}{16}''$ and weighs 10.5 pounds per lineal foot, which is near enough for all practical purposes. The assumed thickness of half an inch used in determining A′ gives a slight error on the side of safety. Laying out the sections of the flanges to scale, the distance between their centres of gravity is found to be a little over 27″, so that in assuming $d = 26.5''$ a slight error on the side of safety was committed.

The difference is so small as not to necessitate a re-calculation.

We can either let the bottom plate extend over a little more than the middle half of the stringer or calculate its theoretically correct length as follows. If in the equation

$$M = \frac{w\,x}{2}\,(l - x)$$

we substitute for w and l their values and for M the greatest allowable bending moment on the beam at the end of the bottom plate, we can solve for x and thus find the length of plate by the equation

$$l' = l - 2\,x$$

The effective area of the lower flange at the end of the plate is about

$$\frac{6 \times 10.5}{10} - 0.77 = 5.53 \,\square'',$$

which multiplied by 4 gives 22.12 tons as the permissible stress on the bottom flange, and this multiplied by the effective depth $27'' = 2.25'$ gives 49.77 as the permissible bending moment. Substituting this gives

$$49.97 = \frac{1.875}{2}\,(21\,x - x^2)$$

$$\text{or} \quad x^2 - 21\,x = -\,\frac{99.54}{1.875} = -\,72.4 \quad \text{nearly}$$

$$\text{therefore} \quad x = \frac{21}{2} \pm \sqrt{-72.4 + \left(\frac{21}{2}\right)^2} = 4.23$$

Consequently the length of the plate l' is $21 - 8.7 = 12.3$ feet: but it is better to increase it to 14 feet.

To find the rivet spacing for the upper flange angles let us divide the beam into lengths of two feet commencing at both ends and moving towards the middle, and let us transform the equation

$$M = \frac{wx}{2}(l - x) = ATd \qquad \text{to}$$

$$AT = S = \frac{wx(l - x)}{2d}$$

and substitute for w, l and d their values, and for x, 2, 4, 6, 8 and 10, S being the actual flange stress at the point considered.

Making the substitutions gives for the various values of S, 11.6, 20.8, 27.5, 31.8 and 33.6 tons. Subtracting each from the one following gives 11.6, 9.2, 6.7, 4.3 and 1.8 tons as the horizontal stresses to be taken up by the rivets in the different two feet lengths.

In addition to these horizontal stresses there is in each length a vertical stress caused by the two ties pressing on the angle irons. What the amount of this vertical stress is it would be hard to say, for the stiffness of the rails tends to distribute the pressure of the wheels over several ties, but if we assume that two thirds of the weight on one wheel or 4.17 tons is supported by the *inner* angle in a two feet length, we will provide for a sufficiently unfavourable case.

From Table XVIII we find that the working bearing pressure for a $\frac{13}{16}''$ rivet on a $\frac{3}{8}''$ plate is 1.829 tons, and that the working bending moment for a rivet of that diameter is 0.395 inch ton.

The total bearing pressure on the rivets in the first length is $\sqrt{(11.6)^2 + (6.25)^2} =$ 13.2 tons nearly, which divided by 1.829 gives 8 as the number of rivets required for bearing. The stress on the inner flange is $\sqrt{(5.8)^2 + (4.17)^2} = 7.15$ tons, and the lever arm is $\frac{1}{2}\left(\frac{11}{16} \times \frac{3}{8}\right) = \frac{17}{32}''$, making the bending moment $7.15 \times \frac{17}{32} = 3.8$ inch tons, which divided by 0.395 gives 10 as the number of rivets required to resist bending: this corresponds to a rivet spacing of 2.4″, or about three diameters.

At the fifth division the stress on the inner angle is $\sqrt{(0.9)^2 + (4.17)^2} = 4.3$ tons, and the corresponding bending moment $4.3 \times \frac{17}{32} = 2.28$ inch tons, which divided by 0.395 gives 6 as the number of rivets required, corresponding to a rivet spacing of 4.″ A larger spacing would do for the lower flange, but it is scarcely worth while to make any difference between the spacings of the upper and lower flanges. The change in the spacing from the end of the stringer to the centre should be made abrupty between stiffeners and not gradually: this is to facilitate the punching by machinery.

Generally speaking it is unnecessary except in the case of very shallow girders to calculate the rivet spacing for the flanges, because the designer may rely upon his previous experience, but, if he be in doubt, he will break no rule of good designing by putting in a few extra rivets.

The stiffeners may be made of $2\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ angle irons, and there should be eight or nine pairs of them. The filling plates will have to be $\frac{11}{16}'' \times 2\frac{1}{2}.''$

In a similar manner may be designed any plate girder or floor beam.

CHAPTER XIII.

THEORY AND PRACTICE OF PIN PROPORTIONING.

The subject of "bridge pins" is one deserving of more consideration than has been accorded it by engineers, and authors of technical works. Until 1878, when Mr. Charles Bender, C.E., presented his paper on "Proportions of Pins used in Bridges" to the American Society of Civil Engineers, very little was known concerning it; the usual custom among engineers when proportioning pins having been to allow one square inch of pin area for every eight or ten thousand pounds of shear in the section most subject to shearing-stress. As Mr. Bender states generally, and as will be shown farther on to be true for iron bridges, it is not the shear, but the bending-moment, which causes the greatest tendency to rupture; so that in any iron structure it will be sufficient, in finding the sizes of pins, to calculate the greatest moment induced in them by the various members coupled thereon, and to proportion accordingly, due regard being paid to the stresses in the eye-bar heads. Before making any investigations, it will be well to review and summarize the most important results of the investigations of others in this subject.

The principal conclusions arrived at by Mr. Bender are, that, for a well-fitting pin of large diameter, a pressure on the bearing-surface of six tons per square inch is not too large; that for simplicity it is well to assume that this pressure is uniformly distributed over the diameter of the pin; that wrought-iron, after millions of impacts, may break on the side where the stress is tensile, but never on the side where it is compressive, the ultimate resistance to crushing being about thirty tons per square inch; that the shearing-stress at the centre of a pin is one and three-eighths times the average shear on the whole section; that in iron and steel the ratio between the greatest allowable tensile and the greatest allowable shearing-stresses should be as 5 to 4, which would make the uniformly distributed shear 2.91 tons per square inch, to correspond with a tensile stress of 5 tons per square inch; and that, owing to various considerations, iron in pins may be strained much more than similar iron in tension members.

Mr. B. Baker, C.E., in "Beams, Columns, and Arches," treats of pins merely incidentally. He finds, that, for iron in solid circular beams, the average value of ϕ is $\frac{11}{16}f$, where f is the ultimate resistance per square inch to rupture by tension, and ϕ the difference between the apparent ultimate resistance per square inch to rupture by bending and f, according to the equation $F = f + \phi$, F being the apparent ultimate resistance per square inch of the extreme fibre which first gives way; and, that for steel, the value of ϕ varies between $1.7f$ and $1.9f$.

Professor Burr devotes five pages of his work on "Stresses in Bridge and Roof Trusses" to the subject of pins, and illustrates the particular case of a suspension-bridge cable pin, and a general case for ordinary truss-bridge pins.

Professor Du Bois, in "Strains in Framed Structures," also gives a mathematical discussion of how to find the maximum bending-moment.

Table XIV. gives the working bending-moments on all the iron and steel pins, and the working-shear on all the steel pins, which will ever be required. Having calculated the bending-moment, the requisite diameter for the pin can be found by looking down the proper column until a bending-moment at least equal to the one found is reached. The diameter will be found at either end of the horizontal row thus located. The use of the column for shear will be made apparent presently.

The upper and lower horizontal lines in the table of bearings (Table XV) give the diameters of the pins; the extreme vertical lines, the necessary widths of bearing-surface at each end of the pins, including both channel and re-enforcing plates; and the other vertical lines, the permissible pressure, on the bearings. The method of using these tables is the following. The pressure which the pin is to carry is to be taken from the diagram of stresses. A trial diameter is then assumed. The vertical column, headed by this diameter, is to be followed down, until a number nearest the pressure to be carried is found. At either end of the horizontal row thus located will be found the proper width of bearing. Knowing the width of bearing, diameter and pressure, the moment to which the pin is subjected may be at once calculated. Turn, then, to Table XIV, and see if this moment agree with the working-moment corresponding to the trial diameter. If it does, all right: if not, another trial is to be made, with a new assumed diameter. After a little experience, the first trial will be sufficient. A consideration of other details, such as widths and depths of eye bars, etc., will frequently aid very much in these trials.

Table XV can be used for bearings in members of lateral systems, portal bracing and vertical sway bracing by multiplying the calculated stresses by two thirds.

To find the least value of the ratio of the diameter of pin to depth of eye bar in an iron bridge, by considering the tension in the bar, and the pressure between the pin and bar,—

Let

w = width of bar,
d_1 = depth of bar,
d = diameter of pin,
C = intensity of working compressive stress,
T = intensity of working tensile stress;

then

$$wd_1T = \text{tension in bar},$$

and

$$wdC = \text{compression on pin and eye.}$$

These, of course, are equal; and, as $C = 6$ tons when $T = 5$ tons, there results the equation,

$$d = \tfrac{5}{6}d_1 = 0.833d_1,$$

which shows that the diameter of the pin should never be less than eighty-three per cent of the depth of the bar. It is possible, though, that good iron of twenty-five tons tensile strength will resist more than thirty tons per square inch in compression: consequently d may be taken at $0.8d_1$ as a matter of convenience.

To find the proportion between width and depth of bars for the smallest allowable pin in an iron bridge,—

Let the notation be as before, and first let us suppose that there be but one pair of bars acting at each end of the pin, and that the total tension be a fixed quantity. The stress in one bar is wd_1T, and its moment is w^2d_1T. This must be equal to the resisting-moment of the pin, which is given by the well-known equation.

$$M = \frac{RI}{D}.$$

Here $R = \frac{3}{2}\ T$, $I = \frac{1}{4}\pi r^4$, and $D = r = \frac{d}{2}$, substituting which gives

$$M = \tfrac{3}{64}\pi Td^3.$$

Equating the two values of the moments gives

$$w^2dT_1 = \tfrac{3}{64}\pi Td^3,$$

or

$$w^2 = \frac{3\pi}{64} \cdot \frac{d^3}{d_1}.$$

Now, to make the diameter of the pin as small as possible, the moment of the stress must be made as small as possible; and, as the stress is constant, the lever-arm w must be made as small as possible. But the product of w and d_1 is a constant: so when w is smallest, d_1 must be greatest. But the greatest value of d_1 is $\frac{5}{4}d$; substituting which gives

$$w^2 = \frac{3\pi}{64} \times \frac{64}{125} d_1^2 = 0.754\, d_1^2,$$

and

$$w = 0.274\, d_1,$$

or about one-fourth of the depth of the bars.

If there be two pairs of similar bars acting at each end of the pin, instead of one pair, the equation of moments will be

$$2\, w^2\, d_1\, T = \tfrac{3}{64}\, \pi\, T\, d^3,$$

or

$$w^2 = \frac{3\pi}{128} \cdot \frac{d^3}{d_1}.$$

As before, to make d a minimum, w must be made a minimum, or d_1 a maximum; therefore $d = \frac{4}{5}d_1$, which, substituted, gives

$$w = 0.194d_1,$$

or about one-fifth of the depth of the bars.

For three pairs of similar bars at each end of the pin, the equation of moments will be

$$3w^2d_1 T = \tfrac{3}{64} \pi T d^3,$$

substituting in which $\frac{4}{5}d_1$ for d gives

$$w = 0.159d_1,$$

or about one-sixth of the depth of the bars.

Finally, if there be four pairs of similar bars at each end of the pin, the equation of moments will be

$$4w^2d_1 T = \tfrac{3}{64} \pi T d^3,$$

which gives

$$w = 0.137d_1,$$

or about one-seventh of the depth of bars.

To find the greatest working shearing-stress (supposed to be uniformly distributed) in terms of the working resistance to tension,—

Let S = actual varying resistance to shearing, considered uniformly distributed. The greatest value of S will correspond to a value of w equal to $0.274d_1$; for suppose the moment to remain at its maximum value, and the dimensions of the bar to vary (consequently the stress therein also), the tension in the bar will be greatest for the value of w corresponding with the greatest value of d_1: therefore the shear will also be greatest for that value.

Equating the tension to the shear gives

$$wd_1 T = \frac{\pi d^2 S}{4}$$

Substituting $\frac{5}{4}d$ for d_1, and $0.274 (\frac{5}{4}d)$ for w, gives

$$0.274(\tfrac{5}{4} d)^2 T = \frac{\pi d^2 S}{4},$$

and

$$S = 0.545T;$$

for $T = 5$ tons, $S = 2.725$ tons. But the greatest allowable value for S is, according to Bender, 2.91 tons. This proves, that, if an iron pin be properly proportioned for crushing and bending, it will be strong enough to resist shear, and in fact, that, before the pin could shear, it would either break by bending or crushing, or the eye of the bar would give way. A similar investigation for steel bridges, where $T = 8.35$ tons, $C = \frac{7}{4} T$, and R (the intensity of working bending-stress) $= 1.8T$, gives $d = 0.5714d_1$, $w = 0.1816d_1$, and $S = 5.912$ tons = the actual intensity of shearing-stress when the pin is strained up to the bending-limit, and the ratio $\frac{w}{d_1}$ for that condition of stress is at its minimum, and consequently the area of the bar, the tension therein, and the shear on the pin, at their maxima. But the greatest allowable shear is, according to Bender, $\frac{4}{5} \times \frac{8}{11} \times T = \frac{32}{55} \times 8.35 = 4.858$ tons; so

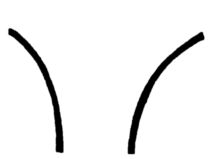
that, for a pair of steel bars pulling on a steel pin in opposite directions, or a single steel bar against a steel bearing, the pin in certain cases will be liable to rupture by shearing, and will therefore have to be porportioned to resist that stress.

After making out the diagram of stresses, and proportioning the main members of a bridge, comes the determination of the sizes of the pins,—a matter that is liable to occupy more time than did all the previous work. Knowing the sizes of all the bars in the structure, the clear width between the inner faces of the top chord channels (and consequently that between post channels) can be found, after which the arrangement of all the bars in the bridge can be decided on. Care must be taken in performing the latter, that no two consecutive chord bars or ties coupled on the same pin pull in the same direction, unless this arrangement reduce the bending-moments, as it can sometimes be made to do; that the lighter set of bars be so placed as to reduce the bending-moments; and that the diagonal ties be placed close to the posts, and the beam hangers close to the ties. Especial care is needed at the panel point where the number of chord bars is different in the consecutive panels. It is possible to arrange the bars there, so that there will be an extremely large moment produced, or so that it will be smaller than at any other panel point of the bottom chord. The neglect of any of these precautions will cause an undue bending-moment on the pin.

The arrangement completed, the next questions to be decided are, first, under what condition of loading will each pin take its greatest bending-moment, and, second, at what point on the pin will this be found. In well proportioned railroad bridges; the bottom chord pins are subjected to their greatest bending-moments when the bridge is fully loaded. Under this condition, the stresses in the chord bars can be taken from the diagram of stresses; but those in the main diagonals must be calculated for the load covering the whole bridge, and their horizontal and vertical components be ascertained.

After having had some practice, one will very often be able by simple inspection to decide at what place the greatest moment of flexure will exist; but, if not, it will be necessary to calculate the values of both horizontal and vertical moments at different points, and find where their combined result is a maximum. As Professor Burr shows, the actual moment is represented by the diagonal of a rectangle whose sides represent the vertical and horizontal moments. It is usually more convenient to square the component moments, add the results, and extract the square root of the sum, than to make out a diagram.

The moment of the stresses can be easily recorded by drawing two curved lines, as shown in the accompanying diagram, representing the directions in which the stresses tend to bend the pin, and writing each moment as calculated under one or other of them, according to whether it would produce positive or negative rotation. The difference between the sums of each column will give the actual horizontal or vertical moment as the case may be.



As a rule single beam hangers and large single counters are to be avoided on account of the great bending moments which they produce upon the pins.

The size of the pin for the hip joint depends greatly upon the arrangement of the bars which it couples. In a double intersection bridge where there are two hip verticals, two long main diagonals and two short ones meeting at the hip, the best arrangement is to put one pair of diagonals on the outside of the chord and the other pair inside, close to the bearing, the verticals coming next and being kept apart by a filler. If the moment on a hip pin be very great, the use of a steel pin will prove advantageous in reducing the size of the eye-bar heads.

Except when the chord pins are small it is not necessary to consider the effect upon them of the stresses in the lateral rods, but whenever possible the latter should be so arranged that the effect of the stress in the outer one will be to diminish the horizontal component of the moment on the pin caused by the stresses in the truss members, *i. e.* if the tendency of the chord and web stresses is to bend the pin convex to the middle of the span, the outer rod, when bent eyes are employed, should point towards the middle; but, if it be to bend the pin concave to the middle of the span, the outer lateral rod should point towards the nearer pier or abutment.

The ends of pins have to be reduced in diameter, so that the nuts and pin pilots may be screwed thereon. Care must therefore be taken in proportioning small pins to see that sufficient area be left under the root of the thread to resist the tension on that section caused by the greatest transverse components of the stresses in the lateral rods. The principal objection to the use of large pins is not always the undue weight of the pins themselves, but the increased size of the chord and tie-bar heads, and the room that they take up.

On the other hand, it is not always desirable to use the smallest possible pin, as the width of the bearing is an inverse function of the diameter of the pin: so if, owing to the necessity of a large number of rivets, the re-enforcing plates be long, it might be economical to increase the diameter so as to reduce the width. Thickening the heads of eye bars has an injurious effect on the pins, although a beneficial one upon the heads, for the lever arms of the stresses are thereby increased.

Bridges with weak pins will not necessarily fail by the rupture of the pins. The reason for this is thus stated by Professor Burr: "The distortion of the pin beyond the elastic limit will relieve the outside eye bars of a large portion (in some cases, perhaps all) of the stress in them. This result will produce a redistribution of stress in the eye bars, by which some will be understrained, and the others correspondingly overstrained. Thus, although the pin may not wholly fail, the safety of the joint will be sacrificed by the overstrained metal in the eye-bars."

The preceding portion of this chapter may be termed the *theory* of pin proportion and the subsequent part the *practice*.

The ordinary method of pin proportioning is to figure the diameters of a few principal pins, and to make the others of the same sizes. Thus, by inspection, can be found which pin near the middle of the bottom chord is subjected to the greatest bending-moment. If there be an even number of panels in the span, it will be the middle pin; but, if there be an odd number, it may be the first or second pin from

the middle, according to the number and arrangement of the chord bars. The vertical component of the bending-moment on any one of these pins is so small in comparison with the horizontal component, that it may be neglected. For bridges with an even number of panels,—

Let

T = tension in middle panels of lower chord,

and

w = the *average* thickness of chord bars in these panels;

then, approximately,

$$\frac{Tw}{2} = \text{bending-moment on middle pin.}\ *$$

This formula may be applied, but perhaps with less accuracy, to a bridge having an odd number of panels; and, if the chord be properly packed, the error will be upon the side of safety.

With the exception of the chord pins at the shoes and at the first panel points from the ends of the span, all the lower chord pins may have a diameter corresponding to this maximum bending-moment, unless the bridge be a long or very heavy one, when some of the intermediate pins may have their sizes determined either by calculation or by interpolating; taking care in the latter case that they be liberally proportioned; for the strength of a pin reduces rapidly with the diameter.

To find the size of the lower chord pin at the first panel point, use the formula,

$$H = \frac{Tw}{2}$$

for the horizontal component of the moment, and the formula

$$V = \frac{tA\,(d + d')}{4}$$

for the vertical component; t being the intensity of working-stress for the hip verticals, A their area (S. R.), to be taken from Table VII, d the diameter or thickness of a hip vertical, and d' that of a beam hanger.

The moment given by the formula

$$M = \sqrt{H^2 + V^2}$$

applied to Table XIV will determine the diameter required. This diameter may be used also for the pin at the shoe.

Where a bottom chord is composed of a continuous strut and eye-bars, the stress on the former cannot affect the pin, for it has no lever arm, consequently in proportioning any bottom chord pin *except that at the shoe* for such a case the value of T is to be found by multiplying the sum of the areas of all the chord bars in the panel considered by the intensity of working stress for those bars.

To find the size of a hip pin, lay off the stresses in one hip vertical and one

* For very long span double track bridges this formula will give an excessive diameter, in which cases the arrangement of the chord packing must be relied on to reduce the bending moments.

end main diagonal to any convenient scale, and find the value of their resultant by the parallelogram of forces. This resultant will determine the thickness of the bearing, a trial diameter being first assumed. It is possible that this bearing will have to be increased, so that there will be enough iron to transfer the stresses from the batter brace, hip verticals, and diagonals, to the chord, as will be explained in the next chapter. An approximate test of the sufficiency of the bearing in this respect may be obtained as follows:—

Let

A = the area of the section of the end panel of the top chord,
d = depth of chord channels,
t = thickness of web of an end chord channel;

then the bearing should not be less than that given by the formula

$$B = \frac{A}{2d} + t.$$

Next find the distance l between the centre of the bearing of the chord and that of the diagonal, also the distance l' between the former and that of the hip vertical, the latter being on the inside. Calling the stress in the hip vertical F, and that in the diagonal S, the vertical moment will be Fl', and the inclined one Sl. Next lay out these components to any convenient scale in their proper directions, and find their resultant by the parallelogram of moments. This resultant will determine the diameter of the pin.

If the diameter found agrees with the one assumed, or if it does not agree, provided that the bearing was not determined by the trial diameter, all right; but if the bearing were so determined, and the two diameters do not agree, another trial must be made.

Where there are more than two main diagonals coupled at the hip, as is the case in double-intersection and in heavy single-intersection bridges, one pair is coupled on the outside of the bearing, and the other on the inside; so that theoretically the greatest bending-moment is equal to the stress in the outer bar multiplied by the distance between the centre of the bar and the centre of the bearing. But practically the moment may be greater, for the distribution of stresses among the diagonals may not be as assumed: so it is well to determine the moment by imagining the outer bar not to exist, and proceeding as explained above for the case of only two main diagonals at the hip, excepting, of course, that the thickness of the bearing must be ascertained by finding the resultant of the stresses in the two diagonals and the hip vertical.

To calculate the size of an intermediate upper chord pin, the widths of chord and post bearings are to be determined as shown in the next chapter. The former is given approximately by the last formula, where A is the section of the panel of the chord on the side of the pin towards the middle of the bridge, and t the thickness of the corresponding channel. The other is given by the formula

$$B = \frac{A_1}{h},$$

where A_1 is the area of the section of the post, and h the depth of one of its channels.

Next resolve one-half of the diagonal stress vertically and horizontally into P and P' respectively. Let l represent the distance between the centre of the diagonal and that of the extension plate, and l' the distance between the former and that of the chord-bearing; then

$$V = Pl,$$

$$H = P'l',$$

and

$$M = \sqrt{H^2 + V^2}.$$

If the bridge be a small one, it will be necessary to calculate only the size of the pin at the top of the first vertical post from the end of the bridge, and to make all the intermediate top chord pins of the same size. But, if the bridge be a large one, it will be better to calculate the diameter of the pin on the post midway between the end vertical post and the middle of the span, and to make all the pins between these places of this diameter, and all the others of the same diameter as that at the end of the first vertical post. After the diameters of the top chord pins are determined, the post and chord bearings should be tested by applying Table XV, although in most cases they will be found ample.

In double-intersection bridges, where the diagonals are halved, and coupled on pins passing through the middle of the posts, the size of any one of these pins may be found from the moment

$$M = \frac{Sw}{2},$$

where S is the stress on the diagonals as given on the diagram of stresses, and w the width of one of the main diagonals.

In all pin proportioning it must be kept in mind that the diameter of the pin is never to be less than eight-tenths of the depth of the deepest bar coupled thereon.

The moment on any pin belonging wholly to a lateral system or sway bracing can always be found by the formula

$$M = Pl$$

where P is the reaction at *one* bearing and l the distance from the centre of this bearing to the centre line of the force which produces P. It must not be forgotten that only one set of diagonals of a lateral system can be in tension at once, and that the stress on any diagonal (where single diagonals are used) should be divided equally between the bearings, making P equal to half the greatest working stress on the diagonal including initial tension.

The value of l and consequently that of M can be reduced by making the eye of square iron and welding it to the rod.

The author wishes to call attention to the superiority (in his opinion) of the simple method given for proportioning lower chord pins by formula over the apparently more accurate one previously explained.

In the former method, when the proper proportion of width to depth of bars is

adhered to, the diameter of the pins will be almost eight-tenths of the depth of the bars, and will be great enough to resist the bending-moments produced by any legitimate method of packing. Moreover, after the diameters of the pins have been determined, the chord can be packed, if it be advisable, so as to reduce the bending-moments. This superabundance of strength in the pins is obtained at the expense of a slight increase in the weight of iron; and the increased sizes of heads for diagonals can do no harm, because they do not enter any limited space, as do the heads at their other ends.

But if, by a skilful arrangement of the packing, we can so reduce the bending-moments on the pins, that the diameters may be made small, and the proportion of width to depth of bars larger than that found in the last chapter, the pins may not be as strong as we imagine them; for we cannot be sure that all the bars are going to pull as we have assumed that they will. It may be that one of the outer bars is a trifle long, and will not pull at all until the others are well stretched: what, then, becomes of our calculated bending-moments?

Any one of them may be so greatly exceeded, that the pin will be strained beyond the elastic limit, and will bend perceptibly, so changing the distribution of stress in the panel that one or more of the bars also may be strained beyond the elastic limit.

But if the pin be large enough, or more than large enough, it cannot bend perceptibly: consequently the distribution of stress will be much more uniform, even if the bars be of slightly unequal lengths.

CHAPTER XIV.

PROPORTIONING OF OTHER DETAILS.

The sizes of stay plates used at the ends of systems of latticing or double-riveted lacing are given in Table XXII. and the sizes of those used at the ends of systems of single-riveted lacing, in Table XXIII. The headings of these tables fully explain their use.

Stay plates are to be employed at the middle of posts (see Pl. IX, Fig. 8) when the diagonals are halved, and connected by pins passing through the posts; their sizes being taken from the before-mentioned tables. Stay plates, if they can be so called, are also to be used on the lower portal struts, for the purpose of attaching the knee braces.

Pin bearings are sometimes figured, counting in both re-enforcing plates and web; but the latter is often omitted. This would be necessary when the holes in the web are bored independently of those in the re-enforcing plates, for then it is very improbable that the different holes will coincide; but, when the re-enforcing plates are riveted to the web before boring, such a precaution is not only unnecessary, but is a waste of material.

By consulting Table XVI. can be found at a glance, accurately enough for all practical purposes, the thickness of web of any Union Iron-Mills channel bar, when the weight is given, or *vice versa*.

Where re-enforcing plates act also as splice plates, there should be when practicable one on each side of the web in order to insure a good, substantial joint.

The length of a simple re-enforcing plate depends upon the number of rivets required, and is thus determined. Find, by dividing the stress given on the diagram of stresses between the various thicknesses of iron which constitute the bearings, the amount of stress which the plate considered is to carry. It is well, though, to make a liberal allowance, say twenty per cent, for the possibility that the stress may not be divided proportionately to the thicknesses. Next multiply the stress so obtained by the perpendicular distance between the central plane of the re-enforcing plate and

that of the plate or web re-enforced; the product will be the moment of the stress upon the re-enforcing plate. Divide this moment by the working bending-moment, taken from Table XVIII, for a rivet of the diameter to be employed for the connection: the quotient will be the number of rivets required to resist bending. Next find, from the same table, the working bearing-stress for one of the rivets upon a plate of the thickness of the re-enforced plate or web, and divide it into the stress which the latter carries: the quotient will be the number of rivets required to afford sufficient bearing. The greater of the two numbers thus obtained is the one to be employed. Next make to scale a drawing of the re-enforcing plate, laying out the rivets, if it be possible, symmetrically, and thus determine the length of the re-enforcing plate. In case of a re-enforced pin hole, if the diameter of the hole exceed one-half the width of the plate, it will be necessary to put more rivets in front of the pin hole than behind it; the ratio of the number in front to the whole number being equal to that of the diameter of the hole to the width of the plate.

The method of proportioning splice plates or connecting plates is somewhat similar. For instance, let us take the plates at a joint in the top chord; which joint, for reasons stated in Chapter IV, is always to be placed a few inches to that side of the pin hole farthest from the middle of the span. The stress on the portion of the plates to this side of the joint is that due to the stress in the panel where the joint occurs; while that on the other portion of the plates is due to the stress in the next panel towards the middle of span. The number of rivets on each side of the joint will be dependent upon the stresses carried by the channel bars of the two adjacent panels, which stresses are most readily determined by multiplying the area of the channels by the intensity of working stress given in Table VIII, and by which they were proportioned. The stress on each channel is to be divided equally or otherwise between the two connecting plates, and the number of rivets on each side of the joint is to be determined in the same manner as for re-enforcing plates.

To determine the length of the cover plate, find in the same manner the number of rivets *upon each side of the joint*, which will take up the stress carried by the chord plate, which stress is to be found by multiplying the area of the section of the top plate by the same intensity as in the last case.

At the hip joint the section of the connecting plates must answer two requirements; first, their area (neglecting, on account of its being bent, the effect of the cover plate) must be sufficient to transfer to the chord a stress equal to that in the first panel; and second, that the pin bearing be sufficient for the resultant of the tensions in the diagonals and verticals meeting at the hip. The length of the cover plate at the hip cannot be calculated, for it carries no stress, simply adding to the rigidity of the joint, and keeping the rain therefrom.

The area of the greatest section of the connecting plate at one side of the shoe made by a plane perpendicular to the direction of the batter brace should be equal to the area of one batter brace channel, or greater if the shoe pin require greater bearing than this would afford; and there should be enough rivets to transfer the stress from the batter brace channel to the connecting channel or plate. Should the batter-brace channels bear against the shoe plates, as they ought to do, there

will be more rivets than necessary; but such a bearing should not be counted upon.

The rules for proportioning shoe, roller and bed plates are given in Chapter VI.

At the intermediate strut connection, there should be enough rivets used in respect to bending and bearing to transfer the calculated stress upon the strut to the connecting plates.

The method of determining the dimensions and number of rivets for extension plates on the upper ends of posts is similar to that explained for splice or connecting plates.

The thickness of the re-enforcing plates at the lower end of a post is determined by the bearing required; and their length in the manner already described. The reason for cutting away the bottoms of the post channels is merely to pack the chord more closely, and thus reduce the bending moments on the pins. But, if the method of pin proportioning recommended be adopted, the necessity for cutting away the channels, to any extent, vanishes; for at the middle of the span the web stresses are so small, that their moments are neglected, and the pins at the feet of the other posts have an excess of strength.

When, because of their large diameter, the lateral rods cannot be attached to the chord pins, but must be connected by vertical pins passing through the lateral strut jaws, they must be made to pull on the middle point of each of the latter pins by using a double eye on one of the rods, with a space between large enough to admit the eye of the other rod. This is to avoid all tendency to rotate the lateral strut about its axis. The rods can be retained in place by fillers above and below.

With this detail, there is a tendency to break the jaw through the pin holes, because of the moment of the longitudinal component of the lateral rod stress: the jaw plate must therefore be made wide enough to properly resist this moment. The easiest way to proportion the plate is to assume its dimensions, and to find its resistance to bending, neglecting the area lost by the pin holes (which area is close to the neutral surface), and making up for the omission by providing a little extra resistance.

To illustrate the method, let us take a two-inch lateral rod, making an angle of forty-five degrees with the planes of the trusses, and let the distance between centres of pin bearings be six inches. The stress on such a rod is $3.14 \times 7.5 = 23.55$ tons, and the bending-moment on the pin is $\frac{1}{2} \times 23.55 \times 3 = 35.3$ inch tons, corresponding (*vide* Table XIV) to a diameter of three inches and a fourth. The distance from the axis of the pin to the centre of the jaw bearing will be about $1\frac{3}{8}'' + 2'' + 1'' + \frac{5}{8}'' = 5''$. The longitudinal component of the stress on the lateral rod is $23.55 \times 0.7 = 16.5$ tons, making the moment on the jaw about $5 \times 16.5 = 82.5$ inch tons. The thickness of the jaw plate should be $\frac{5}{8}''$, and let us assume the width to be $7''$. The resisting-moment is given by the well-known formula,

$$M = \frac{RI}{d_1},$$

where $R = 11.25$ tons, $I = \frac{1}{12} bd^3 = \frac{1}{12} \times \frac{10}{8} \times (7)^3$, and $d_1 = \frac{7}{2}$. Substituting, gives

$$M = \frac{11.25 \times \frac{1}{12} \times \frac{10}{8} \times 49 \times 7 \times 2}{7} = 115 \text{ inch tons, nearly.}$$

The difference between 115 and 82.5, or 32.5 inch tons, is greater than the resisting-moment of the material lost by the pin hole: so the dimensions assumed are ample.

A similar calculation is necessary at the portal rod connection to portal struts. It is evident that the pin holes just treated should be placed as near the ends of the strut as circumstances will permit, in order to reduce the bending moments on the jaws.

It is not customary to calculate the thickness of a beam hanger plate, but to make it from an inch to an inch and an eighth: it can, though, under certain assumptions be calculated. If the load on a plate be considered uniformly distributed over the portion between the beam-hanger holes, and if the flange of the beam be supposed to take up no bending-stress, the plate may be considered as a beam supported at the ends, and uniformly loaded. For instance take the case of a twenty-foot panel of a single track bridge; the reaction at each end of the beam is about 18.5 tons.

Suppose the centres of the beam hanger holes to be situated on the corners of a $4\frac{1}{2}'' \times 6''$ rectangle, the latter dimension being transverse to the bridge, and that the sides of the plate are $8''$ and $9''$, then the bending moment is

$$M = \tfrac{1}{8}\,Wl = \tfrac{1}{8} \times 18.5 \times 4.5 = 10.4 \text{ inch tons.}$$

The resisting moment is $\frac{RI}{d_1}$, where $R = 4$ tons, $I =$ moment of inertia $= \frac{1}{12}\,bd^3 = \frac{3}{4}\,d^3$ and $d_1 = \frac{d}{2}$. Equating the moments, substituting and solving, gives $d = 1.3$ inches.

But as the flanges do assist in resisting the bending, and as some of the weight comes upon the plate outside of the beam hangers, it is safe enough to take the thickness as low as an inch and an eighth.

Lacing, or, as it is often improperly termed, single latticing, is about the most common detail for keeping pairs of channel bars in line: nevertheless, it must be inferior to latticing, especially when the lattice bars are riveted together at their intersection. By inspecting Tables XXII. and XXIII. it will be seen that a system of lacing-bars with one rivet at each end of a bar requires much larger stay plates at the ends than does a corresponding system of latticing or double-riveted lacing.

The actual sizes of lattice or lacing bars for any strut can be determined only by experiment: it is thought that those given in Tables XX. and XXI. are so strong, that the struts on which they are employed would break in the channels rather than in the bars, and yet not so heavy as to cause much unnecessary use of material. It will be seen also in these tables, that the requisite dimensions of latticing and lacing bars depend not only upon the sizes of the channels which they connect, but also upon the distance apart of these channels: this is due to the fact that the bars are subject to compression as well as to tension. The lengths and weights of latticing and lacing bars can be found from Table XIX. It must not be forgotten that these lengths are to be used for *estimates only*; as they were obtained from a diagram, and not checked by calculation.

CHAPTER XV.

DOUBLE TRACK BRIDGES.

For reasons given in Chapter I, special attention has hitherto been given to single track bridges, but as the Japanese engineer may sometime be called upon to design a double track bridge, there will be given in this chapter, although they may have been previously mentioned, the principal differences between bridges for single track and those for double track roads.

In the first place, of course, double track bridges are wider and their live loads twice as great as for corresponding single track bridges. This causes the weight of the track stringers and floor system proper to be doubled, and a large increase on the size and weight of the floor beams. The live and dead load stresses on the trusses are about doubled, thus necessitating in many cases the abandonment of rolled channels for the top chords and batter braces.

The total wind pressure per lineal foot is increased because the area of the vertical projection of one truss is greater, and as the trusses are farther apart the lengths of all the members of the lateral systems and sway bracing are increased, consequently the weights of these portions of the structure are doubly augmented.

For reasons given in Chapter IX, the stresses in the vertical sway bracing are greatly increased. The wind pressure need not be considered to affect the stresses in the chords or posts, for in the first place *ceteris paribus* the wind stresses both direct and indirect are reduced by the increase of width of bridge, and in the second place they are relatively less important by reason of the doubling of the live and dead load stresses. For the same reason stiffened bottom chords are not required in double track bridges.

In these few particulars only does the designing of double track bridges differ from that of single track bridges, and the author is confident that anyone who has thoroughly studied the latter and perfected himself therein will have no trouble whatsoever with the former.

CHAPTER XVI.

ECONOMY.

The first point to be considered, when deciding upon the style of bridge for a certain stream crossing, is the number of spans. It is, in reality, a consideration of economy which determines this; for the best bridge to build, provided that the water-way be not too much contracted, is the one for which the sum of the cost of superstructure and the cost of foundations is a minimum. If the water-way be too much interrupted, the design would not be an economical one, even if its first cost were the least, because of the risk of washout to which the bridge would always be subject.

In most cases there is not much choice concerning the number of spans, local considerations often determining it; but there is occasionally a choice between two or even three numbers. The only way, then, to decide is to make a rough estimate of the cost of the superstructure and the foundations for each number; then, if the choice fall about equally between two numbers, it is better nearly always to adopt the longer spans, because the actual expense for the foundations usually exceeds the amount of the preliminary estimate.

The spans in this country at river crossings are in the author's opinion altogether too short considering the sudden rises and the immense volumes of water in the mountain torrents.

The recent washouts in the neighbourhood of Kioto will give force to this statement.

The next economic consideration is that of depth of truss. Upon this subject much has been written, and many investigations have been made; the general conclusion being, that the depth should be from one-seventh to one-tenth of the span: some English writers say from one-tenth to one-fourteenth of the span; while only one, as far as the author knows, — Benjamin Baker, Esq. C. E., in his treatise on "Beams, Columns, and Arches," — makes it from one-fifth to one-seventh of the span.

Such investigations being purely mathematical, and involving the use of the differential calculus, are of little practical value, as they cannot take into account the numerous variables that ought to be considered. Not only do the stresses in a truss vary with the depth, but also the intensities of working-stress in the compression members. These, again, vary with the number of panels; and this variation is according to a law or laws altogether too complicated to be handled by the calculus. Again: the intensity of working-stress varies, or should vary, according to the position and importance of the member.

In view of the complexity of the question, and wishing to determine the most economic depths for Pratt and Whipple trusses, the author, a year or two ago, undertook to solve the problem in a practical manner by figuring out a number of diagrams of stresses, and bills of materials. At first he considered that it would be necessary to calculate the total actual cost for every case, but upon further investigation found that it would be sufficient to figure out the sections and weights per lineal foot of the different members of one truss, multiply these by their respective lengths, and sum up the products, neglecting all consideration of details, because the differences in the weights of the latter balance each other. Thus, if the depth of a truss be increased by one foot, there would be a little increase in the weights of the lattice bars and rivets and a decrease in that of the pins and eye-bar heads. These may be taken as balancing each other, without making any appreciable error.

The most economic length of panel was at the same time investigated, and was determined, without preparing complete bills of materials, by considering only those portions of the structure which are affected by the variation in the number of panels.

Economy in pony trusses is an element which ought seldom to influence the design, for a good bridge of this kind will generally require more iron than the ordinary calculations demand. Instead of trying to avoid a little expense, regard should be paid to obtaining a good distribution of plenty of material, in order to partly compensate for the lack of rigidity which is characteristic of the pony truss. In very wide pony-truss bridges, especially when the length of span approaches its superior economic limit, it might be well to make a few calculations concerning the economic depth; but the number of panels should be regulated by the slope of the batter braces, which should never be less than two horizontal to one vertical.

The superior economic limit of the pony truss is not a fixed quantity, but decreases as the width of the bridge and the load increase.

After making out diagrams of stresses, and bills of materials, for over one hundred spans, the author came to the following conclusions:—

That if the economic depth be calculated for any span, where the panel length is in the neighbourhood of twenty feet, and if the economic depth for the same span, but with one panel less, be calculated, the latter will exceed the former by one or two feet.

The principal objections to the use of the double intersection for short spans are, that, as the rods are long and slender, they will vibrate more than the shorter and larger ones of the single intersection. Any flaw in a small rod will have a proportionately greater injurious effect than the same sized flaw in a larger rod. Long

for the top chords and batter braces, and if the *sections alone* would indicate a saving of say three hundred pounds of iron by the use of the twelve-inch channels, the others would be more economical; for the twelve-inch channels require larger stay plates, lattice bars, and re-enforcing plates, besides a wider top chord plate, which would increase the weights of the cover plates, chord pins, post latticing, post stay plates, shoe plates, etc., and even add a little to the lengths of the floor beams.

CHAPTER XVII.

BILLS OF MATERIALS, AND ESTIMATE OF COST.

In making out bills of materials, the list of members given in Chapter III. will prove of great assistance. By its use, one can avoid an underestimate due to an omission of any of the parts of the structure. A good way to make out a bill of material is to prepare six vertical columns, in the first of which write the name of the member; in the second, the number of pieces; in the third and fourth, the dimensions determining their section ; in the fifth, their length ; and in the sixth, the weight of all the pieces, or, if of wood, the number of feet, board measure, that they contain.

The following example will serve to explain the method : —

BILL OF WROUGHT-IRON.

Chord channels	12	7"	10½# [	22'	2,772#
Batter-brace channels ...	8	8"	12½# [	33'9"	3,375"
Plate	1	½"	12#	262'	2,620"
Post channels	8	5"	6½# [	22'	1,144"
Lateral struts	4	4"	6# [	15'	360"
Lateral struts	4	5"	6½# [	15'	390"
Main diagonals...	8	¾"	1½"	34'	1,020"
Counters.	8	¾"	¾"	35'	525"
Etc.	—	—	—	—	—

It is to be noticed that it is often conveneint as in the case of the "Plate" to combine several lengths into one.

To the length of each chord bar, main diagonal, and hip vertical is to be added three feet to allow for the weight of the heads; and to that of each adjustable rod about five feet, for the heads, upset ends, and sleeve nuts or turn buckles. Should greater accuracy be required for the weight of an adjustable rod, it will be necessary to ascertain what length will be needed at each end for the heads, and how much for the upset ends and adjusting-nuts by the following

TABLE OF EQUIVALENT LENGTHS OF RODS FOR UPSET ENDS, NUTS, SLEEVE NUTS, AND TURN BUCKLES.

½" —1"	1 upset end and 1 nut	1¼ feet of rod
1 1/16"—1½"	1 upset end and 1 nnt	1⅜ feet of rod
1 9/16"—2"	1 upset end and 1 nut	1⅝ feet of rod
2⅛" —2½"	1 upset end and 1 nut	1 7/16 feet of rod
½" —1¼"	2 upset ends and 1 sleeve nut	2⅜ feet of rod
1 5/16"—2½"	2 upset ends and 1 turn buckle	3 feet of rod

These equivalent lengths do not include the lengths of the upset ends themselves: they represent simply the extra lengths to be added to the bar to equalize the weight of the nuts, sleeve nuts, or turn buckles, and the extra iron for enlarging the ends, which are six or eight inches long.

It is not necessary in a preliminary estimate to find the exact quantities of materials, so approximations to actual dimensions can be made. This will be fully illustrated in the next chapter.

The following table, taken from Carnegie's "Pocket Companion," will be found useful in preparing bills of iron, as will also many of the tables given in Chapter II.

WEIGHT OF RIVETS, and ROUND HEADED BOLTS WITHOUT NUTS, PER 100.

Length from under head. One cubic foot weighing 480 lbs.

Length. Inches.	3/8" Dia.	1/2" Dia.	5/8 Dia.	3/4" Dia.	7/8" Dia.	1" Dia.	1 1/8" Dia.	1 1/4" Dia.
1 1/4	5.4	12.6	21.5	28.7	43.1	65.3	91.5	123.
1 1/2	6.2	13.9	23.7	31.8	47.3	70.7	98.4	133.
1 3/4	6.9	15.3	25.8	34.9	51.4	76.2	105.	142.
2	7.7	16.6	27.9	37.9	55.6	81.6	112.	150.
2 1/4	8.5	18.0	30.0	41.0	59.8	87.1	119.	159.
2 1/2	9.2	19.4	32.2	44.1	63.0	92.5	126.	167.
2 3/4	10.0	20.7	34.3	47.1	68.1	98.0	133.	176.
3	10.8	22.1	36.4	50.2	72.3	103.	140.	184.
3 1/4	11.5	23.5	38.6	53.3	76.5	109.	147.	193.
3 1/2	12.3	24.8	40.7	56.4	80.7	114.	154.	201.
3 3/4	13.1	26.2	42.8	59.4	84.8	120.	161.	210.
4	13.8	27.5	45.0	62.5	89.0	125.	167.	218.
4 1/4	14.6	28.9	47.1	65.6	93.2	131.	174.	227.
4 1/2	15.4	30.3	49.2	68.6	97.4	136.	181.	236.
4 3/4	16.2	31.6	51.4	71.7	102.	142.	188.	244.
5	16.9	33.0	53.5	74.8	106.	147.	195.	253.
5 1/4	17.7	34.4	55.6	77.8	110.	153.	202.	261.
5 1/2	18.4	35.7	57.7	80.9	114.	158.	209.	270.
5 3/4	19 2	37.1	59.9	84.0	118.	163.	216.	278.
6	20.0	38.5	62.0	87.0	122.	169.	223.	287.
6 1/2	21.5	41.2	66.3	93.2	131.	180.	236.	304.
7	23.0	43.9	70.5	99.3	139.	191.	250.	321.
7 1/2	24.6	46.6	74.8	106.	147.	202.	264.	338.
8	26.1	49.4	79.0	112.	156.	213.	278.	355.
8 1/2	27.6	52.1	83.3	118.	164.	223.	292.	372.
9	29.2	54.8	87.6	124.	173.	234.	306.	389.
9 1/2	30.7	57.6	91.8	130.	181.	245.	319.	406.
10	32.2	60.3	96.1	136.	189.	256.	333.	423.
10 1/2	33.8	63.0	101.	142.	198.	267.	347.	440.
11	35.3	65.7	105.	148.	206.	278.	361.	457.
11 1/2	36.8	68.5	109.	155.	214.	289.	375.	474.
12	38.4	71.2	113.	161.	223.	300.	388.	491.
Heads.	1.8	5.7	10.9	13.4	22.2	38.0	57.0	82.0

Before considering a bill of material as finished, it is well to look it over to see that no mistake has been made in the number of the pieces. It is not an uncommon error to put down only half the correct number.

As soon as the bills of iron and lumber are made out and checked, the dead load per foot should be calculated, to see if it agree with the one assumed within the limit specified in Chapter VI.

Estimates of cost should be liberal; for, as a rule, the actual profits on bridges fall short of the amounts estimated. They can be made very readily by using a blank similar to the following: —

Estimate on .... Bridge across ....

..

Length span,............ft. Height,............ft. Clear Roadway,................ft.
Static Load per lineal ft.,...................... lbs. Moving Load per lineal ft., ... lbs. Weight of engine..................lbs on..................wheels
No. of Panels .... Length of Panels ....ft.

		yen	*sen*
Wrought-iron, lbs.	@		
Cast-iron, lbs.	@		
Lumber, ft.	@		
Piles, ft.	@		
Hauling, ...*loads*	@		
Freight	... ...		
Framing	... ...		
Falsework	... ...		
Erection	... ...		
Painting	... ...		
Blacksmithing	... ...		
Coal	... ...		
Freight on tools	... ...		
Travelling expenses	... ...		
Men's time travelling	... ...		
Engineering expenses	... ...		
Teaming during construction	... ...		
Incidentals	... ...		
Total cost of bridge	... ...		
Cost per lineal foot...	... ...		

On fair country roads a day's work for one man may be averaged at 500 pounds drawn 5 ri, that for a horse 1600 pounds drawn 5 ri and that for a bullock 3300 pounds drawn 4 ri: this allows for the time lost in returning with the empty carts*

The designing of falsework will be treated in Chapter XXII. Its cost will include that of the piles in place, if any be required, and that of the lumber, to which should be added about two yen per thousand for framing and raising, and a yen per thousand for taking down. Falsework timber can generally be sold for something when the bridge is completed: so a reduction may be made in its cost when the estimate is to be a close one.

---

* These data have been obtained through the courtesy of Takanobu Kōno, Esq. M. E. Assistant Engineer, Tokio-Takasaki R'y.

The cost of erection can be found approximately for ordinary conditions from the following table. It must not be forgotten that there is a great variation in the cost of erection; for it depends upon the locality, weather, skill of laborers, efficiency of foreman, etc. Those who feel inclined to question the correctness of this table should make some allowance for the difficulty which the author has experienced in getting any data whatsoever upon the subject.

Span.	Yen.	Span.	Yen.	Span.	Yen.
60'	60	150'	330	240'	1010
70'	70	160'	420	250'	1100
80'	110	170'	480	260'	1200
90'	130	180'	550	270'	1300
100'	150	190'	620	280'	1410
110'	170	200'	690	290'	1520
120'	200	210'	770	300'	1630
130'	240	220'	850		
140'	280	230'	930		

In the following table will be found approximately what it ought to cost to give three good coats of paint to bridges of the different spans.

Span.	Yen.	Span.	Yen.
60'	40	200'	300
80'	70	220'	350
100'	100	240'	410
120'	125	260'	470
140'	160	280'	530
160'	200	300'	600
180'	250		

The data, from which this table was made, were taken from the actual cost of painting two Tokio bridges. The contractor's figures were 20 sen per tsubo (36 square feet) for the first coat, the object of which is to prevent rust, 12 sen per tsubo, for the second coat, and 24 sen per tsubo for the third or finishing coat, making in all 56 sen per tsubo.

To the author's American ideas the figures of cost given in the table seem exorbitant. At the time when the Tokio bridges referred to were painted labour was more expensive than it is now, and kinsatsu were cheaper, so it is more than probable that, if present prices were used, the figures in the table would be materially reduced. But as both labour and kinsatsu are always varying, this table will be as good as any other; for by dividing by 56 the cost there given for any particular case and multiplying the quotient by the proper charge per tsubo will be found the probable cost of painting for any number and quality of coats of paint.

In the following table the author has endeavoured to give what he considers ought to be the total actual cost for the single track bridges of this treatise. He has assumed the cost of finished wrought iron at the nearest railway station or seaport to be six sen per pound, that of cast iron four sen per pound, lumber twelve yen per

thousand, falsework two yen per lineal foot of bridge, hauling one yen per ton, erection and painting as per tables, blacksmithing and coal fifteen sen per lineal foot of span, travelling expenses thirty sen per lineal foot, engineering expenses from one to two yen per lineal foot, teaming during construction fifteen sen per lineal foot, common labourer's wages twenty five sen per day, and incidentals ten per cent.

If there be two spans in the bridge, the estimated cost of the two may be reduced by five-per cent., if there be three, the total cost may be reduced by six per cent, and, if there be more than three, by seven per cent.

Span.	Yen.	Span.	Yen.	Span.	Yen.
60'	3,500	150'	12,710	240'	31,000
70'	4,170	160'	14,540	250'	33,450
80'	4,950	170'	16,290	260'	36,150
90'	5,630	180'	17,850	270'	39,450
100'	6,650	190'	19,850	280'	42,500
110'	7,440	200'	21,800	290'	46,300
120'	8,580	210'	24,130	300'	49,500
130'	10,110	220'	26,100		
140'	11,300	230'	28,060		

One paper yen has been taken equal to ninety-five cents in Mexican or Japanese silver, and one dollar or yen of the latter equal to ninety cents gold. The prices used for iron are American. The prices in the table are in paper yen: they should always be changed to suit the varying values of paper and silver money in comparison with gold as a standard.

It must not be forgotten that the estimated costs in the table are for *average* conditions, such as fairly good roads and weather, absence of freshets, a single tier of framed falsework on piles not exceeding twenty feet in length, a favourable location at site for storing materials, erecting tents &c., no probability of a scarcity of labourers or of their being attacked by disease, workmen to provide food and shelter for themselves, no special tools required, and no extraordinary loss of tools or timber.

Before making an estimate on a bridge, one should endeavour to obtain as many as possible of the following.

DATA FOR DESIGNING IRON RAILROAD-BRIDGE SUPERSTRUCTURES, AND ESTIMATING THEIR COST.

Length of span or spans.
Distance of bridge site from nearest railway-station or seaport.
Quality and condition of the roads between these places.
Nature of bed of river, and velocity of stream.
Height of lower chord above bed of river.
Cross section of stream at crossing, showing borings, if any have been made.
Angle which the direction of bridge makes with axes of piers or abutments.

Nature of the country at the site.
Any special difficulty that may be anticipated for the raising.
Kind of falsework it would be advisable to use.
Cost of piles at various places in the neighborhood, if any be required.
Cost of transport of same to site.
Cost of timber per thousand for falsework.
Probable value of falsework timber after bridge is finished.
Cost of withdrawing piles, if necessary.
Number of lineal feet of piles required.
Number of feet of lumber for falsework.
Cost of spikes, bolts, and nails for falsework.
Cost of driving piles.
Cost of transporting pile-driver to and from site.
Common laborer's wages.
Skilled laborer's wages.
Foreman's wages.
Wages for team and teamster.
Cost of superintendence by engineer or engineers.
Number of days' teaming on work.
Date when bridge must be finished.
Probable length of time it will take to raise and complete bridge.
Chances of fair or foul weather during this time.
Chances of having falsework carried away by a sudden rise or an ice-gorge.
Chances of a scarcity of laborers.
Chances of sickness among laborers.
Expenses attendant on same.
Cost of tents or other housing for laborers, if any.
Cost of iron at mill or foundry.
Cost of transport of same to nearest railway-station or seaport.
Cost of lumber per thousand at mill or market.
Cost of transport of same to nearest railway-station or seaport.
Probable expenses for blacksmithing and coal.
Cost of tools, if it be necessary to buy special ones.
Wear and tear of plant, and loss of tools.
Loss of bolts and timber.
Actual cost of raising similar structures under similar circumstances.
Travelling expenses of employees to and from site.
Engineering expenses.
Office expenses in preparing plans, etc.
Advisable allowance for contingencies.

CHAPTER XVIII.

COMPLETE DESIGN FOR A BRIDGE.

Let the bridge to be designed be a through bridge of one span for a single straight track, and let the length of span be 168 feet from centre to centre of end chord pins.

From Chapter VI. we find the live load to be 1150 pounds per lineal foot. Consulting Table I. we see that there should be eight panels, making the panel length 21 feet, that the depth of truss should be 25 feet, and that the probable dead load may be assumed as 1430 pounds per lineal foot.*

In Chapter VII. we find the engine excess for one panel of one truss to be 10.7 tons. Chapter VI. gives the clear roadway about 13.7 feet, and by consulting the diagrams on Plates XXVI. and XXVII. we find the probable width of chord plate to be 20 inches or 1.7 feet, making the width of bridge between central planes of trusses 15.4 feet.

From Chapter VII. we find the wind pressures per lineal foot when the bridge is empty to be about 180 pounds for the upper lateral system and say 310 pounds for the lower lateral system; also $240 + 200 = 440$ pounds per lineal foot for the lower lateral system and say 120 pounds for the upper lateral system when the bridge is loaded.

From Chapter VIII. we find the approximate value of w_4 to be 300 pounds, and that of w_5 310 pounds.

The tangent of the inclination of a diagonal to the vertical is $\frac{21}{25} = 0.84$.

The corresponding secant (vide Table II) is about 1.306, making the length of the diagonal $25 \times 1.306 = 32.65$ feet.

The tangent for the lateral systems is $\frac{21}{15.4} = 1.364$.

From the preceding data we can fill out the following list.

$$n = 8 \qquad \frac{w}{n} = 0.7548$$
$$l = 21 \qquad \frac{w}{n}\sec\theta = 0.9858$$

* It would have been better to assume the dead load to be 1487 pounds per lineal foot. The reason for not having done so is that this chapter was written before Table I was quite finished. The difference being less than four per cent is of no importance.

d	$= 25$	$W_1 \sec\theta$	$= 9.805$
b	$= 15.4$	$\frac{1}{2} W_1 \sec\theta$	$= 4.903$
diag	$= 32.65$	$W'' \tan\theta$	$= 11.379$
$\sec\theta$	$= 1.306$	$\frac{1}{2} W'' \tan\theta$	$= 5.689$
$\tan\theta$	$= 0.84$	$\frac{E}{n}$	$= 1.3375$
$\tan\theta'$	$= 1.364$	$\frac{E}{n} \sec\theta$	$= 1.7468$
W	$= 6.038$	$\frac{E}{n} \tan\theta$	$= 1.1235$
W_1	$= 7.508$	$W_2 \tan\theta'$	$= 4.440$
W'	$= 2.500$	$\frac{1}{2} W_2 \tan\theta'$	$= 2.220$
W''	$= 13.546$	$W_3 \tan\theta'$	$= 6.302$
W_2	$= 3.255$	$\frac{1}{2} W_3 \tan\theta'$	$= 3.151$
W_3	$= 4.620$	$W_5 \tan\theta$	$= 2.734$
W_4	$= 3.150$	$\frac{1}{2} W_5 \tan\theta$	$= 1.367$
W_5	$= 3.255$	$W_6 \tan\theta$	$= 3.661$
W_6	$= 4.358$	$\frac{1}{2} W_6 \tan\theta$	$= 1.830$
E	$= 10.700$		

The next step is to draw the skeleton diagram shown on Plate XIII., numbering the panel points from right to left, beginning with zero at the end. Next by referring to Table III we find the stresses due to the uniform live load, the dead load and the engine excesses on each member of the truss, add them together and write the sum in its proper place on the diagram.

Thus the stress in the second panel of the top chord is

$$7\tfrac{1}{2} W'' \tan\theta + \frac{27}{8} E \tan\theta = 7\tfrac{1}{2} \times 11.379 + 27 \times 1.1235 = 115.677.$$

That in the end main diagonal is

$$\frac{21}{8} W \sec\theta + 2\tfrac{1}{2} W_1 \sec\theta + \frac{11}{8} E \sec\theta$$

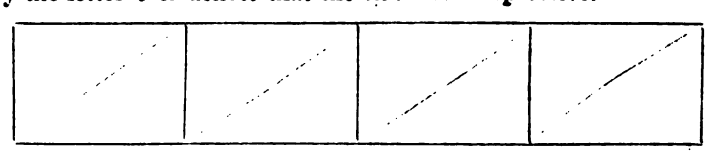
$$= 21 \times 0.9858 + 2 \times 9.805 + 4.903 + 11 \times 1.7468 = 64.430$$

and that in the middle post is

$$\tfrac{6}{8} W - \tfrac{1}{2} W_1 + \tfrac{5}{8} E + W'$$

$$= 6 \times 0.7548 - 3.754 + 5 \times 1.3375 + 2.500 = 9.962.$$

The next step is to find the wind stresses on the windward bottom chord when the bridge is empty: they will be multiples of $W_2 \tan\theta'$, and the coefficients are given in the Table V. There is no need to enter these stresses on the diagram, but they should be written upon a temporary diagram, as in the accompanying figure, followed by the letter C to denote that the stress is compressive.



15.540 *C*	26.640 *C*	33.300 *C*	35.520 *C*
12.813 *T*	12.813 *T*	21.966 *T*	27.457 *T*
2.727 *C*	13.827 *C*	11.334 *C*	8.063 *C*

Thus the stress in the second panel is

$$6\,W_2 \tan\theta' = 6 \times 4.440 = 26.640.$$

The next step is to find the reduced dead load stresses when the bridge is empty: they will be multiples of $W_6 \tan\theta$, and the coefficients are given in the column for W_1 in Table III.

They are to be written on the same diagram under the other stresses, and are to be followed by T to denote that they are tensile. Thus in the second panel the stress is

$$3\tfrac{1}{2}\,W_4 \tan\theta = 3 \times 3.661 + 1.830 = 12.813.$$

The next step is to subtract the values of T from the corresponding values of C, in order to find the greatest actual compressive stresses that can ever come upon the chord. The largest of these differences is 18.827, and it is well to proportion the whole bottom chord to resist this compression, so as to simplify the drawings and shop work. There will be but little waste in so doing, for the different panel lengths of the strut will have to act as tension members, and their effective areas are to be subtracted from the total sections required, when finding the necessary areas for the chord bars.

The next step is to find the wind stresses in the leeward bottom chord when the bridge is covered by the train: they will be multiples of $W_3 \tan\theta'$, and the coefficients are given in Table V.

Thus in the third panel the stress is

$$6\,W_3 \tan\theta' = 6 \times 6.302 = 37.812.$$

The stresses thus determined are to be written on the principal diagram beneath the live and dead load stresses already found.

The next step is to find the stresses in the leeward bottom chord due to the transferred load, when the bridge is covered by the train: they will be multiples of $W_5 \tan\theta$, and the coefficients are given in Table III. under the heading W_1.

Thus in the third panel the stress is

$$6\,W_5 \tan\theta = 6 \times 2.734 = 16.404.$$

The stresses thus determined are to be written under those last found, and the two or three stresses for each lower chord panel are to be added together, the sum being placed beneath.

The next step is to find the sections required for the tension members.

The live and dead load stresses of the bottom chord are to be divided by five, and the combined stresses by seven and a half, and the greater result taken. By inspecting the diagram it will be seen that the combined stresses determine the sections in every case but that of the end panels. The intensities for the main diagonals are 5, 4¾ and 4½ tons, which divided into the proper stresses give the sections required as marked on the diagram.

To proportion the counters we must first decide whether they are to be single or double, then consult Table VI. With channel struts in the bottom chords, single

counters are preferable, although they necessitate rather large pins for the upper chord. Looking down the column for square sections headed "Intensity of Working Stress = 4 tons" in the table we come to a stress of 9.31, the next greater value to 9.746, and following out the horizontal line containing this stress we see that a $1\frac{13}{16}''$ square bar will be required.

Although theory does not call for one, we will put a single 1″ round counter in the third panel, to aid in adjusting the bridge and to assist in taking up shock. By referring to Table VII. we see that two $1\frac{1}{4}'' \times 2''$ bars will be required for the hip verticals.

Next let us proportion the bottom chord strut. The greatest stress was found to be 13.827 tons. It will be necessary to use six inch channels, for the webs of smaller ones would be too much cut up by the pin holes.

The ends of each panel length of strut may be considered fixed, and the number of diameters is $\frac{21 \times 12}{6} = 42$. Consulting Table VIII. we find for these data a working intensity of 2.422 tons, which divided into 13.827 gives 5.7 square inches, corresponding to two 6″ — 9.5* channels. As this member acts as a strut only for wind stresses, it might appear better to employ Table IX., but because it acts also and generally as a tension member, it is better to employ a small intensity of working compressive stress; besides, as said before, any extra material put in the webs of the channels is not wasted, for it will assist in resisting tension.

Referring to Table XVI. we see that the thickness of web of a 6″ — 9.5* Union Iron Mills' channel is 0.3 inch. If we use $\frac{5}{8}''$ rivets for attaching the connecting plates at the joints, the area lost from each channel will be $2 \times 0.3 \times \frac{11}{16} = 0.41\square''$, and from the two channels $0.82\square''$ leaving $2 \times 0.3 \times 6 - 0.82 = 2.78$ say $2.8\square''$ as the effective area of the webs, which area must be subtracted from S. R. in proportioning the bottom chord bars.

By referring to Carnegie's sections of flat bars in Chapter II. we can proportion the main diagonals and chord bars as marked on the diagram. The proportion of width to depth of chord bars should be noticed: it is made as nearly in accordance with the theory of Chapter XIII. as circumstances will premit. For appearance the widths of the main diagonals decrease towards the middle of the span.

Next let us proportion the top chord. From Plates XXVI. and XXVII. we find that 12″ channels must be employed, which makes the ratio of length to least diameter equal to 21.

Referring to Table VIII. we find the intensity of working stress for two fixed ends to be 3.543 tons, which divided into each of the stresses gives the sections required as marked on the diagram.

The width of top plate was assumed as 20″ (we will check it presently to see if it be sufficient), and its thickness should be $\frac{3}{8}''$ (vide Chapter VI.), making the area $\frac{3}{8}'' \times 20'' = 7.5\square''$. Subtracting this from each of the sections required and multiplying each remainder by ten sixths will give the weight of one channel bar for each panel as marked on the diagram.

Next let us proportion the batter braces, for which the ratio of length to least

diameter is 32.65, and the corresponding intensity from Table VIII for two fixed ends is 2.889 tons. Dividing this into 84.623 gives 29.29 square inches as the section required.

Subtracting from this 7.5 square inches and multiplying the remainder by ten sixths gives 36.32 pounds as the weight per foot of each channel.

Next let us proportion the posts.

Assuming 12″ channels for the end ones, makes the ratio of length to least diameter 25, for which Table VIII. gives for two hinged ends an intensity of 2.71 tons. This divided into 37.122 gives 13.7 □″ as the sectional area of the two channels. Multiplying by ten and dividing by six gives 22.83 pounds per foot as the weight of each channel. Consulting Carnegie's channel sections in Chapter II., we find that this section is obtainable.

Assuming ten inch channels for the next post makes the ratio 30, the intensity 2.332 and the section required 9.98 □″, corresponding to two 16.55 pound channels. It will be necessary, however, to use 17.5 pound channels, for the Union Iron Mills roll nothing between this and 16 pound channels. Let us assume six inch channels for the middle post, making the ratio 50, the intensity 1.256 and the section required 7.93 □″, corresponding to two 13.22 pound channels. This is not an economical section, so let us try seven inch channels, making the ratio 42.86, the intensity 1.565 and the section required 6.37 □″, corresponding to two 10.62 pound channels, which we find from Chapter II. are obtainable.

It is now time to look to the chord packing and see that the assumed width of top chord plate is sufficient, yet not too great. Referring to Table VII. we find that the size of the beam hangers is $1\frac{5}{16}$″ square. From Table XVI. we find the width of flange for a 12″—45.* channel is about 3.1″ doubling which and subtracting the product from 20″ leaves 13.8.″ The area of the top chord inner connecting plate should be a little more than half the area of one channel say 7 □″, which divided by 12 gives about 0.6 say $\frac{5}{8}$″ or 0.68″ as the necessary thickness. Subtracting twice this from 13.8 and allowing a clearance of $\frac{1}{8}$″ for the post inside the chord will leave 12.42″ as the distance between inner faces of post channels. Allowing $\frac{1}{2}$″ for the thickness of the jaw plate at the foot of the post will leave 11.42″ as the clear packing space. Into this must go four main diagonals each 1″ thick, two beam hangers and the chord strut. The sum of the thicknesses of the diagonals and beam hangers is 6.63″ to which adding $\frac{1}{2}$″ for clearance and subtracting the sum from 11.42 leaves 4.29″ for the strut. As the latter should be about square, and as the connecting plates on the outside should be half an inch thick, the space required by the strut will be 7″, showing clearly that this arrangement will not suit.

There is no reason why we should not pack the main diagonals outside of the middle post, therefore we will do so, and obtain 4″ more room. At the foot of the next post there is just room enough, for two main diagonals go inside and two outside of the post, and the counter passes between the chord strut channels. The same remarks apply to the feet of the end posts, and as there is abundant room at the upper panel points and at the shoe, we see that the assumed width of 20″ is just right.

This completes the proportioning of the main members of the trusses.

The sizes and weights of the track stringers and floor beams we will take directly from Tables XI. and XII., for the method of proportioning these members was sufficiently exemplified in Chapter XII.

We can either interpolate the sizes of the lateral, portal and intermediate struts, and of the lateral and vibration rods from Table XIII. or determine them more exactly by calculation. For the purpose of exemplification we will adopt the latter method.

Let us begin with the upper lateral system. The panel wind load is

$$\frac{180 \times 21}{2000} = 1.89 \text{ tons},$$

and the inclination of the rods to the struts is about equal to the arc whose tangent is the panel length divided by the clear roadway or

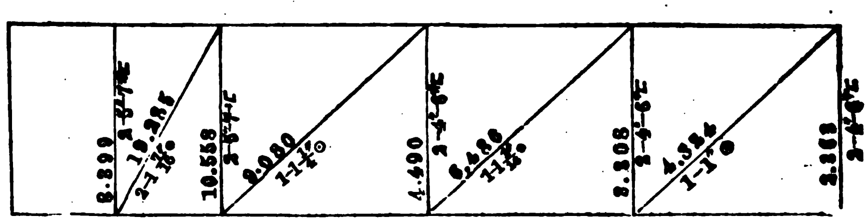
$$\tan^{-1} \frac{21}{18.7} = \tan^{-1} 1.533 \text{ or (vide Table II.) } 56°53'.$$

The corresponding secant is about 1.88 making the length of the diagonal $18.7 \times 1.88 = 25$ feet.

The following is, therefore, table of data

$$n = 8$$
$$W = 1.89$$
$$\frac{W}{n} = 0.2363$$
$$\text{and } \frac{W}{n} \sec \theta = 0.4324$$

Using the formulæ at the beginning of Chapter IX. or employing Table V., we can calculate the stresses in the lateral struts and rods and record them as in the accompanying diagram.

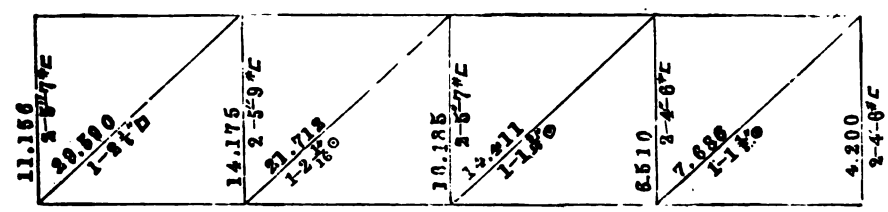


For the lower lateral system we have the moving panel wind load equal to $\frac{240 \times 21}{2000} = 2.52$ tons, and the static panel wind load $= \frac{200 \times 21}{2000} = 2.10$ tons.

The secant will not differ essentially from that found for the upper lateral system. We can therefore fill out the following table of data, the notation corresponding with that of Chapter IX.

$$n = 8$$
$$W = 2.52$$
$$W_1 = 2.10$$
$$\frac{W}{n} = 0.315$$
$$\frac{W}{n} \sec\theta = 0.5764$$
$$W_1 \sec\theta = 3.843$$

Using the formulæ of Chapter IX. or employing Table V. we can calculate the stresses on the lower lateral rods and all the lower lateral struts except those between the pedestals, and enter them upon the following diagram



To find the stress on the strut between pedestals we must employ Equations 5 and 6 of Chapter IX., and adopt the greater of the two stresses thus found.

For Eq. 5, we have

$$W_a = \frac{810 \times 21}{2000} = 3.255 \text{ tons},$$

$$W_b = \frac{180 \times 21}{2000} = 1.89 \text{ tons},$$

$$G = \frac{1430}{2000} = 0.715 \text{ ton},$$

$$G_1 = w_4 = \frac{800}{2000} = 0.15 \text{ ton}.$$

Substituting gives

$$C_n = \frac{15}{4} \times 3.255 + \frac{7}{4} \times 1.89 - \frac{168}{16}(0.715 - 0.800) = 11.156 \text{ tons}.$$

For Eq. 6 we have

$$W'_a = \frac{440 \times 21}{2000} = 4.62 \text{ tons},$$

$$W'_b = \frac{120 \times 21\frac{1}{4}}{2000} = 1.2[illegible] \text{ tons},$$

$$G' = \frac{1150 + 1480}{2000} = 1.29 \text{ tons},$$

$$\text{and } G'_1 = w_5 = \frac{310}{2000} = 0.155 \text{ ton}$$

Substituting gives

$$C_n' = \frac{15}{4} \times 4.62 + \frac{7}{4} \times 1.26 - \frac{168}{16}(1.29 - 0.31) = 9.24 \text{ tons},$$

showing that when the bridge is empty the stress on the end lower lateral strut is greater than when the span is covered by the moving load. The reader must not conclude that such is the case for all spans, as it is probable that the reverse would be true for spans exceeding two hundred and thirty or two hundred and forty feet.

Next let us ascertain the stresses in the vertical sway bracing.

Using the notation of Chapter IX. we can obtain the following data

$$P' = \tfrac{1}{2} \times 25 \times 1 \times 50 \times \frac{1}{2000} = 0.313 \text{ ton}$$

$$P = \frac{90 \times 21}{2000} - \frac{P'}{2} = 0.945 - 0.156 = 0.789 \text{ ton}$$

$$b = 15.4$$

$$d = 25$$

$$f = 10$$

$$\sec\theta = \frac{[(15.4)^2 + (10)^2]^{\frac{1}{2}}}{10} = \frac{18.36}{10} = 1.836$$

The stress in the vibration rod is therefore

$$V \sec\theta = \frac{2 \times 25\,(0.789 + 0.313) - 2 \times 0.313 \times 10}{15.4} \times 1.836$$

$$= 5.824$$

The stress in the intermediate strut is

$$C = \frac{25}{10}\,(0.789 + 0.313) - 0.313 = 2.442 \text{ tons.}$$

For the portal bracing

$$P' = \tfrac{1}{2} \times 32.6 \times 1 \times 50 \times \frac{1}{2000} = 0.408 \text{ ton}$$

$$P = 3\tfrac{1}{2} \times \frac{90 \times 21}{2000} - \frac{P'}{2} = 3.308 - 0.204 = 3.104 \text{ tons}$$

$$P_e = \frac{90 \times 21}{2000} = 0.945$$

$$b = 15.4$$

$$d = 32.6$$

$$f = 13$$

$$\sec\theta = \frac{[(13)^2 + (12)^2]^{\frac{1}{2}}}{13} = \frac{17.7}{13} = 1.36 \quad *$$

The stress in one pair of vibration rods is therefore

$$T = \frac{2 \times 32.6\,(3.104 + 0.408) - 2 \times 0.408 \times 13}{15.4} \times 1.36$$

* It is evident that this method of obtaining $\sec\theta$ is approximate.

$= 19.285$

The stress in the lower portal strut is

$$C = \frac{32.6}{18}(3.104 + 0.408) - 0.408 = 8.399 \text{ tons}$$

and that in the upper portal strut is

$$C' = C + P - P_6 = 8.399 + 3.104 - 0.945 = 10.558 \text{ tons.}$$

The portal stresses have been entered on the diagram for the upper lateral system. Next by referring to Table VI. and using the column for round sections headed "Intensity of Working Stress = 7.5 tons," we can proportion all the lateral and the portal vibration rods as marked on the diagrams, also the intermediate vibration rods, which we find will have to be $1\frac{1}{8}''$ diameter.

To proportion the intermediate strut we must add to the stress found the horizontal component of the initial tension in one vibration rod, which Table VI. shows to be 1.25 tons.

The cosine of the inclination to the horizontal is $\frac{15.4}{18.36} = 0.84$, making the horizontal component $1.25 \times 0.84 = 1.05$ tons and the total stress on the intermediate strut $2.442 + 1.050 = 3.492$ tons.

Consulting Table X. we see that a 4″ I-beam 14′ long will have sufficient strength, but it will be well to use a 5″—11 pound beam, as a 4″beam allows very little room for the connecting plates to fit between the flanges.

Next let us proportion the portal and lateral struts. Before doing so we must add to each stress already found, the longitudinal components of the initial tensions in all the rods meeting at one end of the strut.

For example let us take the second lower lateral strut on which the calculated stress is 14.175 tons. The cosine of the angle which the rods make with strut is $\frac{1}{1.83} = 0.546$. Table VI. gives the initial tensions on a $2\frac{1}{8}''$ square and a $2\frac{1}{16}''$ round rod respectively as 4.128 and 3.125 tons, the sum of which multiplied by 0.546 is 3.960 tons, which added to 14.175 makes 18.135 tons for the total stress. The length of the strut may be taken as 18.7 feet, and the depth of the channels may be assumed as 5 inches, even though larger ones might be more economical, for it is evident from Plate III. that the width of the strut should be kept as small as practicable. These dimensions make the ratio of length to least diameter about 33, for which with one fixed and one hinged end, Table IX. gives 3.445 as the intensity of working stress. Dividing this into 18.135 gives 5.27 square inches as the section required, corresponding to two 8.78 pound channels. Consulting Carnegie's sections in Chapter II., we find that no such channel is rolled, so it will be necessary to employ 9 pound channels.

The sections of all the other lateral and the portal struts are found in a similar manner, and are written both on the diagram of this chapter and on the principal diagram (Plate XIII.), as are also the sections of the portal and lateral rods. The sections of the intermediate struts and vibration rods are marked on the principal diagram upon the end vertical post.

Comparing the sections of the members given in Table XIII. with those found in this chapter we do not in all cases find an exact agreement. This is because the table was made before the exact wind pressures per lineal foot were determined, nevertheless with one exception the agreement is sufficiently close. The exception is in the sizes of the upper lateral rods, which the table gives as greater than those found in this chapter. This was done advisedly in preparing the table, in order to insure sufficient lateral stiffness to the bridges and thus prevent undue vibration under passing loads. It is only for spans under 200 feet that this allowance is necessary.

It is now a convenient time to test the size of the middle post to see if it be strong enough to resist the bending and transferred load stresses due to a wind pressure of thirty pounds per square foot, in addition to the live and dead load stresses.

Chapter IX. gives the stress produced by bending as

$$\frac{(P + P')(d - f)}{2m} = C$$

Here P and P' have values equal to $\frac{120}{180}$ of those used when calculating the stresses in the vertical sway bracing, while the values of d and f are the same as before. The value of m is nearly 1.1 feet. The following is then the table of data

$$\begin{aligned} P + P' &= 0.735 \\ d &= 25 \\ f &= 10 \\ m &= 1.1 \end{aligned}$$

Therefore $C = \frac{0.735 \times 15}{2 \times 1.1} = 5$ tons (nearly).

Using an intensity of five tons makes the section required for one channel, to resist bending, 1.00 square inch.

The simplest way to find the value of the transferred panel load V is to multiply the panel length by the wind pressure per lineal foot, multiply the product by the depth of truss and divide by the perpendicular distance between centres of trusses, reducing the result to tons.

Thus $V = \frac{21 \times 120 \times 25}{15.4 \times 2000} =$ about 2 tons.

Using the intensity for which the post was proportioned viz. 1.565 tons, and remembering that V must be equally divided between the two channels, we find the additional area of *each* channel, necessary to resist the stress considered, to be 0.64 square inches, making the total area of one channel, needed to resist the effect of wind pressure, $1.00 + 0.64 = 1.64$ square inches. This is slightly greater than one half the area of one channel, or 1.59 square inches, as previously determined, but the difference is so small that we will conclude that the post is strong enough.

Next let us proportion the pins, beginning with the middle one of the bottom chord.

The value of T in the formula $M = \frac{Tr}{2}$ of Chapter XIII. is most easily obtained

Next let us calculate the size of the pin at the top of the end vertical post. The chord bearing is given by the equation

$$B = \frac{32.6}{2 \times 12} + 0.75 = 2.1'' \text{ say } 2\tfrac{1}{8}''$$

and the post bearing by the equation

$$B = \frac{13.7}{12} = 1.14'' \text{ say } 1\tfrac{1}{8}''$$

These quantities make the lever arms

$$l = \tfrac{1}{2}\,(\,1\tfrac{1}{8} + \tfrac{11}{16}\,) = \tfrac{29}{32}'' \text{ say } 1'' \text{ to allow for play,}$$

and $l' = \tfrac{1}{2}\,(\,2\tfrac{1}{8}'' + \tfrac{11}{16}\,) + 1\tfrac{1}{8} = \quad ''$ say $2\tfrac{5}{8}''$ to allow for play.

We must again suppose the outer bars with their stresses not to exist, therefore the vertical and horizontal components of $\frac{45.2}{4} = 11.3$ tons, determined graphically to be respectively 8.6 and 7.3 tons, are the stresses to be considered as producing the bending.

The vertical and horizontal component moments are therefore respectively

$$V = 8.6 \times 1 = 8.6 \text{ inch tons}$$

and $H = 7.3 \times 2\tfrac{5}{8} = 19.2$ inch tons,

making the resultant moment

$$M = \sqrt{(19.2)^2 + (8.6)^2} = 21 \text{ inch tons,}$$

corresponding to a diameter of $3\tfrac{1}{2}''$.

As there are but two diagonals in the next panel, the stress on one will be 13.5 tons; and as the lever arms are but slightly greater in this case than in the last, we can obtain the resultant moment approximately by multiplying the preceding value by $\frac{13.5}{11.3}$ or 1.2, making the required value 25.2 or say 29 inch tons to allow for the slight increase in the lever arms. This corresponds to a $3\tfrac{1}{4}''$ pin, which dimension will not only be adopted for this panel point, but for convenience also at the end of the outer vertical post.

The same diameter may be adopted for the middle pin, or its proper size may be determined in a similar manner.

The total stress on the counter is 9.746 tons plus the initial tension of 3.334 tons, as given in Table VI., or 13.080 tons.

The half of this stress, 6.5 tons, passes to each side of the post and to each side of the chord, or would do so if a double eye were used on one counter. The vertical and horizontal components of this stress determined approximately by scale are 5 tons and 4.2 tons. The corresponding lever arms may be roughly taken as 5 inches and six inches respectively, making the component moments

$$V = 5 \times 5 = 25 \text{ inch tons}$$

and $H = 6 \times 4.2 = 25.2$ inch tons

The resultant moment is about

$$M = 25\sqrt{2} = 35.3 \text{ inch tons}$$

corresponding to a $3\frac{1}{4}''$ pin. As the horizontal components of the initial tensions may be taken as balancing each other, the calculated value of H is too high, so we may conclude that a $3\frac{1}{4}''$ pin will be strong enough. Had the counter stress been much larger, it would have been necessary to use double counters close together at their feet and spread apart at their upper ends, so as to reduce the size of the middle top chord pin, because it is bad practice to let the single counters pull eccentrically upon the pin.

The portal pins may be proportioned by assuming the lever arm of the stress on a portal rod to be $1\frac{1}{8}''$, and the stress (vide Table VI.) $10.298 + 1.875 = 12.173$ tons, making the moment approximately $\frac{9}{8} \times 12.17 = 13.7$ inch tons, corresponding to a $2\frac{3}{8}''$ pin, for the column for lateral pins is to be used in this case.

The stress at each bearing of the vibration rod connection to the upper lateral struts is (vide Table VI.) $\frac{1}{2}(6.205 + 1.250) = 3.73$ tons; and the lever arm may be assumed as $2\frac{1}{4}''$, making the moment $2\frac{1}{4} \times 3.73 = 8.39$ inch tons, corresponding to a $2''$ pin.

The diameter of the bolt for connecting the vibration rod to the intermediate strut may be calculated; but, if it be assumed to be $1\frac{1}{4}''$, it will have abundant strength, provided that the round iron of the eye be properly flattened so as to reduce the bending moment.

Vertical pins will be required for the lateral rod connections at the ends of the three lower lateral struts nearest each end of the span. As the lateral rods are single, they must be attached to the middle of the pins. To reduce the bending moment the centres of bearings must be brought as closely together as possible. The least possible distance is six inches, making the bending moment $3P$, where $2P$ is the greatest working stress on the rod. The several values of $2P$ as found from Table VI. are 33.867, 25.058 and 14.381 tons, making the corresponding moments 50.8, 37.58 and 21.57 inch tons and the corresponding diameters $3\frac{5}{8}''$, $3\frac{1}{4}''$ and $2\frac{3}{4}''$.

We are now ready to proceed with the "Bill of Iron," in making which, close approximations of lengths are allowable.

Let us prepare the blank form recommended in Chapter XVII., then turn to the list of members given in Chapter III., and fill out the form, proportioning as we go any details whose sizes have not been previously determined. The filling-out of the part denominated "Main Portions" is a very simple matter, and needs but little explanation. It is to be noticed that the lengths of the chord bars and main diagonals have been increased by three feet to allow for the weights of the heads, and those of all adjustable rods by five feet to allow for the weight of the eyes, upset ends, and adjusting-nuts.

The grouping of members having some similar dimensions is to be observed. It involves considerable economy of labor if one has to estimate on many bridges. In filling out the last vertical column, the tables of weights of flat, round and square iron near the end of Chapter II. will be found of great assistance.

Let us employ latticing for the top chords, batter braces, posts, and portal struts, and single-riveted lacing for the upper lateral struts, and bottom chord struts.

Referring to Tables XXII. and XXIII., we find the size of stay plates for the top chords and batter braces to be $\frac{3}{8}'' \times 8\frac{3}{4}''$, d being a little greater than D; that for the middle posts $\frac{1}{4}'' \times 7\frac{1}{2}''$, d being nearly equal to $2\ D$; that for the next larger posts $\frac{5}{16}'' \times 8\frac{1}{2}''$, d being less than $1.5\ D$ but greater than $1.25\ D$; that for the largest posts $\frac{3}{8}'' \times 8\frac{3}{4}''$, d being a little greater than D; that for the upper lateral struts $\frac{1}{4}'' \times 7\frac{1}{2}''$, d being a little greater than 1.5 D; that for the 4″ channel lower lateral struts $\frac{1}{4}'' \times 7''$, d being a little less than 1.5 D; that for the 5″ channel lower lateral struts $\frac{1}{4}'' \times 8\frac{1}{2}''$, d being a little less than $1.25\ D$, that for the portal struts $\frac{1}{4}'' \times 5\frac{1}{2}''$, d being a little less than $1.25\ D$; and that for the bottom chord struts $\frac{1}{4}'' \times 9''$, d being equal to D.

To find the thickness of the connecting or reinforcing plates at the hip we must divide the area of the section of the first panel length of top chord by twice the sum of the depths of the outer and inner plates, i. e.

$$t = \frac{26.25}{2\,(12+9)} = 0.625 = \frac{5}{8}''$$

To find the length of each plate beyond the centre of the pin hole, let us assume as an approximation that the total stress on the chord is equally divided between the four plates, making that on each one about 23 tons. The lever arm for this stress is $\frac{1}{2}\left(\frac{5}{8} + \frac{1}{2}\right) = \frac{9}{16}''$ making the moment $\frac{9}{16} \times 23 = 12.94$ inch tons. Let us use $\frac{7}{8}''$ rivets, the working bending moment for one of which (vide Table **XVIII.**) is 0.494 inch ton, which divided into 12.94 gives 27 as the number of rivets required to resist bending. The total pressure on the bearings is about 46.5 tons, and as the web (vide Table XVI.) is nearly $\frac{1}{2}''$ thick the working bearing pressure for one rivet is (vide Table XVIII.) 2.625, making the number of rivets required for bearing

$$\frac{46.5}{2.625} \text{ or } 18$$

showing that for the connecting plates of the top chords the rivets need be proportioned for bending only. By making a rough sketch of the connection it is readily seen that we may use four horizontal rows of rivets two inches apart, and a pitch of 8″ without bringing the centres of rivet holes more closely together than good practice allows. This arrangement will bring the end of the plate about 28″ from the centre of the pin hole. We could make a similar calculation for the plates attached to the batter brace, but we can see that the same number of rivets will suffice, for although the stress carried by each plate is less, the lever arm is greater on account of the greater thickness of the channel web. In order to allow for the bolt hole of the portal strut connection we must add two or three inches to the length of the plate, so if we say that the total length of plate measured along the centre lines of chord and batter brace is 5′, we will be pretty near the mark.

To find the thickness of the connecting plates at the first joint in the top chord we must divide the area of one channel of the third panel by the sum of the widths of an inner and an outer plate. It will therefore be $\frac{25.15}{2(12+9)} = 0.6$: if, however, we

make the inner plate ⅜" thick and the outer one $\frac{9}{16}$", the total area of section will be just right and the plates of procurable sizes. Although one plate has a considerably greater area than the other, we must still consider that one half the stress on the channel is carried by each plate, for such is the probable division. It is impossible to determine the exact method of division, but this one will cause no undue stress on either plate, because the actual intensity of working stress for the plates may be at least one fourth greater than that for the strut itself.

The stress on each plate on the side of the joint nearest the middle of the span is, therefore, about $6.3 \times 3.543 = 22.3$ tons, which multiplied by $\frac{1}{2}$ ($\frac{7}{8} + \frac{1}{2}$) gives 15.33 inch tons as the bending moment upon the rivets. Dividing this by 0.494, the working bending moment for a ⅞" rivet, gives 32 as the number of rivets required on the centre side of the joint.

To find the number required for the other side, we ascertain the stress by multiplying $\frac{18.75}{4}$ by 3.543, making 16.6 tons, multiplying this by $\frac{1}{2}$ ($\frac{5}{8} + \frac{1}{2}$) and dividing the product by 0.494, which gives 19.

By making a sketch of the connection and using the same arrangement for the rivets as at the hip, a plate about four feet long will be required.

A similar calculation for the next joint shows that the inner plate should be ⅝", and the outer one $\frac{11}{16}$" thick and that the length should be about 5′. The thicknesses of the plates for the middle joint will be the same as those last determined, but the length should be a little greater, say 5.5′. All these dimensions of plates are to be properly entered on the bill of iron.

To find the dimensions of a connecting plate for the bottom chord strut, let us make the area of the section of same through the pin hole equal to the area of one channel, or 2.85 square inches. If d be the width of the plate at this point and t the thickness, the effective area will be $t\,(d-4)$. Assuming $t = \frac{1}{2}$, and equating to 2.85 will give $d = 9.7$ say 10″: this can be gradually reduced to 6″ at the ends, but the 10″ depth should extend at least six inches on each side of the panel point for the sake of appearance.

The greatest tensile stress which need be provided for is $5 \times 2.85 = 14.25$ tons. The thickness of the web of a 6″—9.5* ⊏ is given by Table XVI. as 0.3″, making the lever arm $\frac{1}{2}$ $(0.3 + 0.5) = 0.4''$, and the moment $14.25 \times 0.4 = 5.7$ inch tons. Dividing this by 0.18 (the working bending moment for a ⅜" rivet) gives 32 as the number of rivets required on each side of the joint. By making a drawing to scale it can be ascertained that it is just possible to use three rows of rivets 1¼" apart, with a horizontal rivet spacing of 2½". Allowing for the joint and pin hole a distance of 11″ between nearest rivets would make the length of the connecting plate about 6′4″, which is excessive. It would be better to employ two rows of staggered ⅞" rivets, for each channel would then be weakened by only one rivet hole, and there would be fewer rivets needed. The number required is 12 it is found by dividing the moment 5.7 by the working resisting moment 0.494. Using a pitch of 3″ will not bring the rivet holes too closely together: this will make each plate about 4′ long, which is not too great. It will be noticed that we assumed the whole section to be subjected to an intensity of five tons instead of only the effective section, as would

really be the case if the pin holes be properly slotted. The reason for so doing is that, as a general rule, rivetted members should not be subjected to tension, but, if they be so subjected the number of rivets used for the connection should be greater than that called for by theory. Then again, because of the cambre, there is a tendency to straighten out the strut, the bending moment of which must be resisted by the rivets at the joints. As the assumed total tension on the strut is about twice as great as that to which it would be actually subjected, the theoretical number of rivets required has been doubled, which, all things considered, is not making too great an allowance for safety.

We will put a joint at the second panel point from each end of the span, and another at the middle.

For the other panel points reinforcing plates will be required to compensate for the material lost at the pin hole. The greatest area lost is $0.3 \times 4 = 1.2$ square inches, corresponding to six tons of stress, the moment for which is $6 \times 0.4 = 2.4$ inch tons. This divided by 0.494 gives 11 as the total number of rivets on *both* sides of the pin hole, or six on each side. It will be better to make this number eight on a side, for fear that the reinforcing plate be called upon to bear more than its calculated stress. A thickness of $\frac{3}{8}''$ and a maximum depth of 8″ will more than compensate for the material lost in the web, and the lever arm being thus slightly reduced will lessen the bending moment upon the rivets. A drawing to scale will show that the length of each plate should be about 3′. Those at the ends of the span may be made nearly a foot shorter.

Next let us proportion the reinforcing or connecting plate at the shoe.

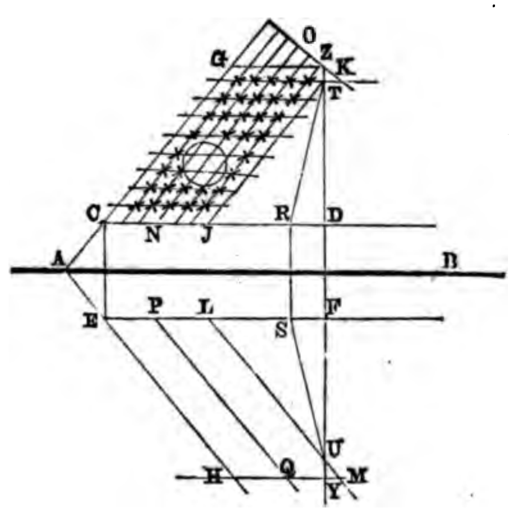
The greatest live and dead load stress at the channel bearing is say $\frac{1}{2}(54.482 - 5 \times 2.8) =$ about 20 tons. Referring to Table XV. we find the necessary bearing for this stress upon a $8\frac{1}{4}''$ pin to be $1\frac{1}{16}''$. Table XVI. gives the thickness of web for a 12″ — 36.82 pound channel as 0.62″, which, subtracted from $1\frac{1}{16}''$, leaves 0.45″ say $\frac{1}{2}''$ as the thickness of plate required. If L be the length of the greatest section of this plate by any plane perpendicular to the batter brace, then $\frac{1}{2} \times L$ should not be less than the area of one batter brace channel. Supposing equality we have

$$\tfrac{1}{2} L = 10.9 \text{ and } L = 21.8''.$$

This condition will receive attention presently; meanwhile we will assume the thickness to be $\frac{1}{2}''$, and calculate the number of rivets required by multiplying the stress on one channel ($10.9 \times 2.889 = 31.5$ tons) by the lever arm $\frac{1}{2}(0.62 + 0.5) = 0.56$, and dividing the product (17.64 inch tons) by 0.494, the resisting bending moment of a $\frac{7}{8}''$ rivet, making the number 35.

For this connection we can use five rows of rivets spaced two inches apart, and a pitch of about three and a half inches so as to bring the rivets into horizontal lines as in the accompanying diagram, which is thus drawn. Lay out a horizontal line A B, and from any point A draw the lines A G and A H, making angles with A B to correspond to the arc whose tangent is $\frac{24}{31}$. Draw C D and E F parallel to A B and at distances therefrom equal to $\frac{1}{2}(20'' - 2 \times 2.85'') =$ about 7″, cutting A G and A H in C and E. Draw N O and P Q parallel respectively to A G and A H, and at distances 6″ therefrom. Draw also the parallels J K and L M 6″ from N O and

P Q. Let us assume that the vertical distance between the centre of the pin hole and the bottom of the shoe plate and that from the centre of a pin hole to the bottom of a post are each ten inches. Subtracting 1″ for the thickness of the shoe plate, let us lay off the remaining 9″ perpendicular to C D and E F so as to determine the centres of the pin holes. Laying off the limiting circle of rivets with an assumed radius of 3½″, we can mark the rivet centres as in the diagram and



determine the height of G Z above C D to be 22½″. If from Z we drop the perpendicular Z Y to A B and thus determine the base of the bent plate to be C D F E we would find it out of proportion and probably too large. Let us see if it be so. The greatest pressure on a leeward shoe is

$$\frac{168}{4}\left(\frac{1480 + 1150 + 810}{2000}\right) + \frac{13}{8} \times 10.7 = \text{about } 78 \text{ tons.}$$

Let us assume the length of each roller to be 20″ and the diameter 2½″, then Table XVII. gives the permissible pressure as 7.91 tons, which divided into 78 gives ten as the number of rollers required.

Allowing a space of 1″ between rollers and a play of three inches would make the total length of shoe plate 37″. This is out of proportion to the width, so let us assume a new width of 25″ and try again. The last mentioned table gives the permissible pressure on a roller 2½″ × 25″ as 9.88 tons, which divided into 78 gives eight as the number required, making the length of shoe plate 30″. This is a better shape and will therefore be adopted.

Allowing the shoe plate and the batter brace upper plate to project 3″ beyond C E in the diagram, will leave 27″ to be laid off from C E to the right hand edge of the shoe plate, R S. We can now join R T and S U, and determine G Z T R S U Y H E C G to be the development of the bent connecting plate. Its dimensions are averaged on the bill of material.

If from R we measure the perpendicular to C G, we find it to be about 20¾″,

which is near enough for all practical purposes to the 21.8″ required by theory.

The next details on the "List of Members" are the reinforcing plates at the feet of the posts. Table XV. gives for a bearing stress of 18.6 tons and a 3⅝″ pin a bearing width of ⅞″, subtracting from which 0.325, the web thickness of a 12″ — 22¼* [, leaves 0.55 as the thickness to be furnished by the outside and inside reinforcing plates, from which it is evident that the least allowable thickness of ⅜″ will be sufficient for each.

Let us assume that half of the 18.6 tons is borne by the inner reinforcing plate, and the remainder by the outer plate and the web. The moment of stress on the rivets will then be $9.3 \times \frac{1}{2}\left(\frac{3}{8}+0.325\right) = 3.255$ inch tons, which divided by 0.311, the working bending moment for a ¾″ rivet gives 11 as the number of rivets required to resist bending.

The total pressure on the rivets cannot exceed

$$18.6\left(\frac{2 \times \frac{3}{8}}{2 \times \frac{3}{8} + 0.325}\right) = \text{about 13 tons,}$$

which divided by 1.46, the approximate working bearing pressure for a ¾″ rivet on a plate 0.325″ thick, gives 10 as the number of rivets required for bearing. It will be better to use a greater number of rivets than the calculations call for, because the reinforcing plates should extend a few inches higher than the lower edges of the stay plates so as to stiffen the otherwise unsupported projecting ends of the post channels. A drawing to scale would show the height required to be about 2′ making the total length of the inside plate 5′, and that of the outer say 2.4′. These dimensions are applicable to the other post reinforcing plates, the widths for which should be equal to the depths of the channels to which they are attached, provided that such depths be not less than 8″. These dimensions are entered on the "Bill of Iron."

Next come the plates for connecting the intermediate struts to the posts.

It is not worth while to make any calculations for this connection, because the stress on the strut is so small.

We will use two bent plates each ⅜″ 4″ × 2.5′ A simple calculation shows that the diameter of the pin for attaching the vibration rod should be 1¾″, which would leave 1 ⅛″ of iron outside of the pin hole, which is sufficient.

Next come the plates for connecting the lower portal struts to the brackets, the dimensions of which we will assume to be $\frac{5}{16}$″ × 9″ × 15″.

Next come those for connecting upper portal struts to name plates, the size for which we will assume to be $\frac{5}{16}$″ × 9″ × 2′.

Next come the connecting plates for track stringers over floor beams, the dimensions for which may be taken as ⅜″ × 8″ × 3′.

Next come the cover plates. The length of those at the hip may be taken as 2′, for they carry no stress that can be calculated.

The stress carried by any intermediate cover plate is $\frac{3}{8} \times 20 \times 3.543 = 26.57$ tons, which multiplied by ⅜ gives 9.96 inch tons as the moment on the rivets. Using ⅞″ rivets we have

The jaw plates of the portal struts act also as reinforcing plates, so their thickness may have to be proportioned for this condition. The pressure on the bearing is equal to the greatest working stress on a $1\frac{1}{16}''$ rod, which *for an intensity of 5 tons* Table VI. gives as 6.240 + 1.875=8.115 tons. The diameter of the pin was determined to be $2\frac{3}{8}$." Consulting Table XV. we find the necessary width of bearing to be $\frac{5}{8}''$, subtracting from which the web thickness $\frac{1}{4}''$, leaves $\frac{3}{8}''$ for the thickness of the plate. As the latter will have to resist bending, it will be well to make the thickness $\frac{1}{2}''$. The width should vary from 8" to 5", making the average say 7". This width strictly speaking should be calculated, but it is hardly necessary; for, judging by a similar calculation in Chapter XIV., we can conclude that if the pin hole be placed as closely as possible to the end of the strut, and if the jaw plates be made 8" wide at the pin holes, there will be sufficient material to resist the bending produced by the transverse component of the greatest working stress on the portal rods. The greatest stress on one channel supposing it to belong to the truss would be 2.1 × 2.488 = 5.12 tons, the intensity being taken from Table VIII. for 84 diameters and one end fixed. The lever arm is $\frac{1}{2}$ ($\frac{1}{2}$ + $\frac{1}{4}$) or $\frac{3}{8}''$, making the moment on the rivets $\frac{3}{8}$ × 5.12 = 1.92 inch tons, which divided by 0,811 gives 7 as the number of $\frac{3}{4}''$ rivets required. A calculation for bearing would give a smaller number. Laying out the detail to scale, we find that the required length of the jaw plate is about 8′.

Next come the extension plates. Let us first proportion those for the largest post. The thickness for one extension is found by dividing the total sectional area of the post by the depth of the channels, or $\frac{13.7}{12}$ = 1.14 say $1\frac{1}{8}''$, which Table XV. shows to be more than sufficient for bearing. This extension plate can be made up of two plates each $\frac{9}{16}''$ thick, the inner one extending two or three inches below the upper edge of the stay plate, and the outer one as low as requisite.

The stress carried by the rivets is $\frac{1}{2}$ × 87.122 = 18.6 tons and the lever arm is $\frac{1}{2}$ ($\frac{9}{16}$ + 0.88) = 0.45", making the moment 18.6 × 0.45 = 8.87 inch tons. Using $\frac{5}{8}''$ rivets we divide by 0.494 and find the number required to be 17. Only one half of those rivets which pass through both thicknesses of plate and the channel web are to be counted in making up the 17; and the countersunk rivets passing through only the two thicknesses of plate are not to be counted at all.

By laying out the connection to scale we see that the inner plate should extend about 10" below the ends of the channels, and that we can use five rows of rivets with a 3" pitch, permitting of the passage of 13 rivets, half of which being deducted from 17 leaves 10 or 11 rivets to pass through the outer plate and the web alone, making the former extend 18" below the top of the post channels. The total length of the outer plate is, therefore, 28", and that of the inner plate 20." Similar calculations for the extension plates of the other posts will give the dimensions recorded in the "Bill of Iron."

The thickness of the shoe plate is given in Chapter VI. as 1", and its other dimensions have been determined to be 25" × 80".

The thickness of the roller plate is also 1", its width about 25 + 2 × 3 = 31", and its length 80 + 2 × 3 + 2 = 88".

The area of the shoe plate is 25″ × 30″ less about 30 square inches for the anchor bolt holes, or 720 square inches which multiplied by 200 and divided by 2,000 gives 72 tons as the greatest permissible pressure upon the shoe plate, if it rests directly on the masonry.

The greatest actual pressure was ascertained to be 78 tons, therefore we must either use bed plates at the fixed end of the span or increase the area of each shoe plate to 780 + 30 = 810 square inches. If we make the width 26″ and the length 32″, we will have 832 square inches of area, and there will be no undue projection of plate beyond the pedestal, therefore this method will be adopted.

The bed plates for track stringers may be made $\frac{3}{4}$″ × 14″ × 14″, and the anchor plates $\frac{1}{2}$″ × 6″ × 8″.

The beam hanger plates may be made $1\frac{1}{8}$″ thick. The distance between centres of beam hangers may be averaged at 9.″ A $1\frac{5}{16}$″ square bar upsets (vide table in Chapter XIX.) to $1\frac{7}{8}$″, for which the greatest diameter of a hexagonal nut is (vide table in same chapter) 4.04″, making the necessary length of plate 13″. The necessary width will be 4″ + $1\frac{5}{16}$″ + 4″ say $9\frac{1}{2}$″. The weight of each of the two name plates may be assumed to be 50 pounds.

Next come the latticing and lacing bars. The spread of those for the top chords and batter braces is about 17″, and the stretch should be about the same. The length of space latticed on each chord panel is about 18′ 6″, and that on each batter brace about 28′ 6″, which distances divided by 17″ and multiplied by 2, show that 26 bars will be required for each panel length of chord and 40 for each batter brace, making the total number 472.

Table XX. makes the size of the bars $\frac{3}{8}$″ × $2\frac{1}{2}$″, and Table XIX. makes the length 2.008 + 0.231 = $2\frac{1}{4}$′ nearly.

In a similar manner are determined the number and dimensions of all the lattice and lacing bars recorded in the "Bill of Iron". For simplicity in shop work we will use stay plates instead of lacing for the lower lateral struts: these will permit of the passage of the stringer connecting plates through the struts.

The lengths of the pins on the "Bill of Iron" include an allowance for the nuts, and a reduction for diminished diameters. The diameter of the anchor bolts is found in Chapter VI. to be $1\frac{1}{4}$″, and their length may be assumed to be 6′.

We will allow 100* for temporary bolts used in erection: these need not be wasted, for they can be saved for erecting another bridge.

An allowance of 150* for ornamental work will be sufficient.

Let us average the length of filler for each pin at 4″, and the weight per lineal foot of filler at 12 pounds or the weight of each filler or pair of fillers on a pin 8 pounds.

For the fillers over end floor beams, we will employ 12″ —42* I divided along the middle of the web, as there is no tee iron deep enough in Carnegie's sections.

For the shoe pin supporting pieces, we will employ 12″ — 60* I beam with the upper flange removed: the length may be averaged as in the Bill of Iron to obtain the approximate weight.

The weight of the stringer bracing frames is included in that of the stringers.

The track rails with their fish plates and attaching spikes do not belong to the bridge, for they would have to be provided if the bridge did not exist, so their weight is not included in the "Bill of Iron", but must be afterwards allowed for in checking the dead load: it may be taken to be 45 pounds per lineal foot. Lock nuts will be used on counters, lateral rods, vibration rods and beam hangers: let us average their weight at $\frac{1}{4}^{\#}$ each. An allowance of 50 pounds will be sufficient for pilot nuts.

The total weight of rivet heads may be calculated from the group in the list of members, but it is not worth while to do so; for, if three per cent, be added to the total weight of iron, it will cover the weight not only of the rivet heads, but also that of the allowance for rivets lost during erection.

The "Bill of Lumber" needs no explanation, except that there is an allowance for an extra tie at each end of the span.

BILL OF IRON.

Top Chord Channels	8	12″	31.25* [	21′	19,873
" " "	8	12″	44.92* [		
" " "	8	12″	45.12* [		
Batter Brace Channels	8	12″	36.32* [	34′	9,879
Post Channels...	4	7″	10.62* [	25.5′	9,311
" "	8	10″	17.5* [		
" "	8	12″	22.83* [		
Up. Lat. Strut Channels	10	4″	6* [	14′	840
Low. " " "	6	4″	6* [	13′	468
" " " "	4	5″	7* [	13.5′	864
" " " "	4	5″	9* [	13.5′	
" " " "	4	5″	7* [	14′	392
Portal Strut Channels	8	5″	7* [	14.2′	795
Bot. Chd. Strut Channels	4	6″	9.5* [	169′	6,422
Intermediate Struts	5	5″	11* [	14.3′	787
Floor Beams *	7	@	3400*	each	23,800
Track Stringers †	8 pairs	@	5100*	each	40,800
Guard Rails	2	5″×4″	14.5* [	172′	4,988
Top Chord Upper Plate	2	⅜″	20″	127′	9,8[illegible]
Batter Brace Upper Plate	4	⅜″	20″	35′	
Main Diagonals	8	1″	3¼″	35.65′	13,841
" "	16	15/16″	3½″		
" "	16	13/16″	4″		
Hip Verticals	8	1¼″	2*	28′	1,867
Chord Bars	24	13/16″	4½″	24′	17,883
" "	24	⅞″	4½″		
" "	16	⅝″	3½″		
" "	8	1 3/16″	3½″		

* Including all details for same.
† Including all details for same and the bracing frames.

Extension Pl., Inner	8	$\frac{9}{16}''$	$12''$	$20''$	550
„ „ „	8	$\frac{9}{16}''$	$10''$	$20''$	
„ „ „	4	$\frac{7}{16}''$	$7''$	$20''$	68
Shoe Plates	2	$1''$	$25''$	$30''$	417
„ „	2	$1''$	$26''$	$32''$	462
Roller Plates	2	$1''$	$31''$	$38''$	654
Tr. Stringer Bed Plates	4	$\frac{3}{4}''$	$14''$	$14''$	163
„ „ Anchor „	8	$\frac{1}{2}''$	$6''$	$8''$	53
Beam Hanger Plates	14	$1\frac{1}{8}''$	$9\frac{1}{2}''$	$13''$	540
Name Plates	2	@	$50^{\#}$	each	100
Lattice Bars, Chds. and B. Br. ...	472	$\frac{3}{8}''$	$2\frac{1}{2}''$	$2\frac{1}{4}'$	3,319
Lacing Bars, Bot. Chd. Str.	1280	$\frac{1}{4}''$	$2''$	$0.6'$	1,280
Lattice Bars, Posts	272	$\frac{3}{8}''$	$2\frac{1}{2}''$	$2\frac{1}{8}'$	1,806
„ „ „	272	$\frac{5}{16}''$	$2\frac{1}{4}''$	$2\frac{1}{8}'$	1,355
„ „ „	136	$\frac{5}{16}''$	$1\frac{7}{8}''$	$2\frac{1}{8}'$	565
Lacing Bars, Up. Lat. Str.	300	$\frac{1}{4}''$	$1\frac{7}{8}''$	$0.85'$	399
Lattice Bars, Port. Str.	228	$\frac{1}{4}''$	$1\frac{1}{2}''$	$1.1'$	314
Pins, Top Chords	4	$3\frac{7}{8}''$	⊙	$2'$	315
„ „ „	10	$3\frac{1}{2}''$	⊙	$2.5'$	802
„ Bot. „	8	$3\frac{1}{4}''$	⊙	$2'$	442
„ „ „	4	$3\frac{5}{8}''$	⊙	$2.8'$	385
„ „ „	6	$4''$	⊙	$3'$	754
„ Low. Lat.	4	$3\frac{5}{8}''$	⊙	$1'$	446
„ „ „	4	$3\frac{1}{4}''$	⊙		
„ „ „	4	$2\frac{3}{4}''$	⊙		
„ Portal	8	$2\frac{1}{2}''$	⊙		
„ Vib. Rod Con. to Lat. Str. ...	10	$2''$	⊙	$0.6'$	63
Bolts, Name Plate	4	@	$1^{\#}$	each	4
„ , Vib. Rod Con. to Int. Str. ...	10	$1\frac{7}{8}''$	⊙	$1'$	92
„ , Portal Struts to B. Br. ...	8	$2\frac{3}{4}''$	⊙	$1'$	158

Int. Con. Plates, Outside	8	$\frac{9}{16}''$	9″	4′	540
„ „ „ „	8	$\frac{11}{16}''$	9″	5′	825
„ „ „ „	4	$\frac{11}{16}''$	9″	5.5′	454
Con. Plates, Bot. Chd. Struts ...	12	$\frac{1}{2}''$	$8\frac{1}{2}''$	4′	680
Rein. „ „ „ „ ...	16	$\frac{3}{8}''$	$7\frac{1}{2}''$	3′	435
„ „ „ „ „ ...	8	$\frac{3}{8}''$	$7\frac{1}{2}''$	2′	150
Con. Plates at Shoes	4	$\frac{1}{2}''$	14″	30″	233
„ „ „ „	8	$\frac{1}{2}''$	21″	$23\frac{1}{2}''$	548
Rein. Pl. at Feet of Posts, Inside ...	4	$\frac{3}{8}''$	12″		
„ „ „ „ „ „ „ ...	4	$\frac{3}{8}''$	10″	5′	650
„ „ „ „ „ „ „ ...	2	$\frac{3}{8}''$	8″		
„ „ „ „ „ „ , Outside...	8	$\frac{3}{8}''$	9″		
„ „ „ „ „ „ „ ...	8	$\frac{3}{8}''$	$7\frac{1}{2}''$	2.4′	456
„ „ „ „ „ „ „ ...	4	$\frac{3}{8}''$	5″		
Con. Pl. Int. Struts to Posts	20	$\frac{3}{8}''$	4″	2.5′	250
„ „ Port. Struts to Brackets ...	4	$\frac{5}{16}''$	9″	15″	47
„ „ „ „ „ Name Pl. ...	2	$\frac{5}{16}''$	9″	2′	38
„ „ Tr. Str. over Fl. Bms. ...	14	$\frac{3}{8}''$	20″	3′	420
Cover Plates, Top Chords	14	$\frac{3}{8}''$	20″	2′	700
Filling Plates, „ „	24	$\frac{1}{8}''$	12″	2′	240
Jaw Pl., Up. Lat. Str., Outer ...	10	$\frac{5}{8}''$	5″	3′	313
„ „ „ „ „ , Inner ...	10	$\frac{1}{2}''$	$4\frac{1}{2}''$	2′	150
„ „ Low. Lat. Str., Outer ...	6	$\frac{5}{8}''$	5″	3′	188
„ „ „ „ „ , Inner ...	6	$\frac{1}{2}''$	$4\frac{1}{2}''$	2′	90
„ „ „ „ „ , Outer ...	12	$\frac{5}{8}''$	$6\frac{1}{2}''$	4′	650
„ „ Port Struts	8	$\frac{1}{2}''$	7″	3′	280
Extension Pl., Outer	8	$\frac{9}{16}''$	12″	28″	770
„ „ „	8	$\frac{9}{16}''$	10″	28″	
„ „ „	4	$\frac{1}{2}''$	7″	28″	109

BILL OF LUMBER.

Shims (Oak)	16	7″	8″	21′	1568
Ties "	138	7″	8″	6′	3864
" "	9	7″	12″	6′	378
" "	24	7″	8″	12′	1344
Foot Planks (Pine)	16	3″	12″	21′	1008
Total Number of feet b. m. ...	...	...	...	...	8162

The total weight of iron, less a small amount belonging to the floor system proper projecting beyond the end pins, divided by 168 gives 1282 pounds as the weight per foot for the ironwork, agreeing very well with the amount found by interpolation from Table I.

The total weight of lumber is $7154 \times 4\frac{1}{2} + 1008 \times 2\frac{1}{2} = 33{,}521$ pounds, which divided by 172 feet, the distance over which the lumber extends, gives 195 pounds as the weight per foot of the lumber. The total weight per foot will therefore be

$$1282 + 195 + 45 = 1522.$$

From this must be subtracted something to allow for the weight of iron that rests directly upon the masonry, such as the heavy pedestals, end lower lateral struts, bed plates, rollers, anchor bolts &c., in all about 5300 pounds weight or 32 pounds per lineal foot, making the proper dead load per lineal foot $1522 - 32 = 1490$ pounds. The assumed dead load was 1430 pounds, making the difference 60 pounds per lineal foot, or just four per cent., which is within the allowable limit of error specified in Chapter VI. It will, therefore be unnecessary to make the calculations anew.

It may appear to the reader who has carefully followed out all the calculations in this chapter, that the designing of iron bridges, and estimating weights thereof, involve a great deal of work, and demand considerable time: but such is not necessarily the case; for an expert could have made this design in from three to four hours, because his experience would have told him the sizes of many of the details and the number of rivets to employ. In this chapter everything has been figured out carefully enough for making working-drawings, instead of merely an estimate of weight; for the author considers that it is better to teach the beginner exact methods in the first place, and leave him to develop approximate ones as his practical experience increases.

CHAPTER XIX.

WORKING-DRAWINGS.

The first points to be determined before commencing a working-drawing are the scale and the size of the paper. The least scale which it is convenient to use is one inch to the foot, and the greatest scale for a whole drawing should seldom exceed an inch and a half to the foot. If a smaller scale than one inch be used, difficulty will be experienced in writing the rivet spacing between the rivet holes. The width of the paper should be from three and a half to four and a half, or even five feet: and, as for the length, it is better to use roll-paper, and not to cut it until the limits of the drawing be determined; for it is a great convenience to be able to make all the working-drawings for a bridge upon a single sheet.

The following is a draughtsman's equipment for making working-drawings in a methodical and expeditious manner: a table from four to five feet wide, from six to eight feet long, and about three feet high; a pair of steps each four or five inches rise, and three feet long; a bevelled steel straight-edge, at least three feet long; a beam compass with tangent screw attachment; a couple of small triangles (rubber ones are the best); some four-H and six-H pencils; a little tracing-paper; a finely divided duodecimal boxwood scale (the subdivisions being quarters, eighths, and sixteenths); a good box of instruments, including a protractor and a pair of hairspring dividers; and the usual outfit of rubbers, tiles, pens, etc., that one finds in draughtsmen's offices. T-squares, large triangles, and parallel rulers should never be used in making a working-drawing. The first can never be depended upon, because of the impossibility of having both board and T-square always perfectly true; no wooden ruler can be relied on not to warp; and parallel rulers are a delusion.

For a few inches it is permissible to turn right angles with triangles, but for long distances the beam compass should be used; and parallel lines can be most accurately drawn by erecting a perpendicular near each end of the original line, and laying off on them equal distances. When distances are a little too great for the triangles, and too small for the beam compass, the large ordinary compasses can be used; but it will be found that they are seldom required. The four-H pencils are to be used for writing dimensions, etc., and the six-H ones for drawing lines. The draughtsman should always have at least one of the latter sharpened to a chisel edge for ruling, and another to a point for sketching. He will find it to be greatly to his advantage to keep his pencils always well sharpened, for an error of the width

of a pencil-line will often cause a great deal of inconvenience. A piece of emery paper or a fine file will be found useful for sharpening pencils. The tracing-paper will be convenient in transferring drawings of similar chord heads, etc.: its function is merely the saving of a little time.

It is generally better to have both a long and a short scale. The long one may be divided into feet only, the inches and fractions of inches being taken from a diagonal or other small scale. If the draughtsman be not provided with a suitable scale, he can easily prepare a very fair one for himself on a strip of the roll of paper upon which the drawing is to be made.

The method of projecting one view of a piece from another view will not do for working-drawings, owing to the liability of the triangles to slip. All measurements should be transferred by the dividers; and, if there be any probability of the points of the dividers having been moved, the distance between them should be tested by laying it off once more upon the original length. There should be no more than a single transferrence of any one distance, for errors often increase, instead of balancing.

The general arrangement of a working-drawing consists merely in laying out a plan and elevation of one-half of the span, leaving at least a foot of space at each end, and six or eight inches above the elevation and below the plan, if there be room to spare, with the same distance, or a little more, between. As it is immaterial if different portions of the drawing cross each other, provided that such intersection cause no conflicting of the measurements, the various members may be shown in several views alongside of their respective positions in plan and elevation.

Thus the top chord may be represented in an under and an upper view above the elevation of the truss, and the batter brace may be shown in a similar manner above and to one side of the elevation. Projections of the posts on planes transverse to the bridge may be drawn alongside and a little below the elevation of these members, the amount of lowering being sufficient to bring the ends of the strut clear of the chords. Attached to the projections of the posts can be shown the intermediate struts and vibration rods, with their connections; and shortened views of the chord bars and diagonals can be placed alongside their elevations in order to represent the heads clear of all other members. Passing to the plan, on one side is drawn the packed lower chord, and attached thereto the lower lateral rods and struts in half-length; while alongside the latter can be represented an elevation of the same with the floor beams beneath, and an end view of the beams near by. The stringers can also be represented in this neighbourhood. At the other side of the plan, can be shown half-lengths of the upper lateral rods and struts in two views, and a projection of the portal bracing on the plane of the batter braces, and on planes at right angles thereto. Each detail can be delineated to any required extent in the neighbourhood of its position in plan, elevation, or both. If necessary, the panel points on one side of the plan may be brought opposite the middle of the panels on the other side, in order to avoid too much intersection.

This arrangement, although a good one, is by no means the only one, and in some cases might not be the best. For instance, in skew bridges it would be well to show the whole of the lower lateral system in the plan, and the whole of the upper

lateral system above the elevation, in connection with the uppermost view of the top chord, which should be the plan from above. Then, again, if the bridge be a large one, the height may be so great that it will be impossible to show the plan below the elevation; in which case it will be necessary either to make separate drawings for the plan and elevation, or to place one alongside of the other on the same sheet. In making tracings of the working-drawing, the tracing-cloth can be shifted about so as to group similar parts and so as to avoid too much intersection of different portions.

Provided that any piece be symmetrical about a plane cutting it at the middle of its length and at right angles thereto, it will be sufficient to show only one-half of the piece; and the measurement may be referred to the end of the member, to the central plane, or to both. Where the same detail is used in more places than one, it is not necessary to show it more than once, provided that it be *exactly* the same in every respect.

As an illustration of how to make a working.drawing, take the case of the bridge treated in the last chapter, and assume that the paper and table are each four and a half feet wide. Using the scale of an inch to the foot, the depth of the elevation will be about two feet two inches, and the width of the plan about one foot five inches. Allowing five inches above the elevation, and as much more between elevation and plan, will bring the lower side of the plan within an inch of the edge of the paper: this arrangement will do very well. The first step is to draw a line with the steel straight-edge, as nearly as possible, without taking too much trouble, paralled to the length of the paper, and at a distance of two feet five inches below the upper edge. This line should be very fine and perfectly straight. It can be made so by prolonging it half the length of the straight-edge at a time, and afterwards testing it in several places. On this line take a point a foot or more from the left-hand end of the paper, as the centre of the end lower chord pin. Lay off along this line with the greatest possible accuracy the panel length, until the centre of the bridge be reached: in this case twenty one feet must be laid off four times. At the panel points erect short perpendiculars with the triangles, and on the perpendicular at the centre lay off the camber, which in this case is three inches (*vide* Chapter VI.). Had the bridge contained an odd number of panels, it would have been necessary to draw the middle panel, and lay off the camber of three inches at each end of this panel. Then, assuming the curve of the chord to be a parabola, the fall from the centre to any panel point is equal to the camber at the centre multiplied by the square of the ratio of the distance of the panel point considered from the middle of the span to the half-length of span.

Thus in the case considered, the falls at the first, second, and third panel points will be respectively $3(\frac{3}{4})^2$, $3(\frac{1}{2})^2$, and $3(\frac{1}{4})^2$, or $\frac{27}{16}''$, $\frac{3}{4}''$, and $\frac{3}{16}''$, making the heights of these points above the horizontal line respectively $3'' - \frac{27}{16}''$, $3'' - \frac{3}{4}''$, and $3'' - \frac{3}{16}''$, or $1\frac{5}{16}''$, $2\frac{1}{4}''$, and $2\frac{13}{16}''$; which distances are to be laid out upon the perpendiculars so as to locate the centres of the lower chord pins. The length of the panels as thus determined differ from those of their horizontal projections by an inappreciable quantity. If there be any lengthening of the chord, it may go against

the play of the pins in the eyes.

Next join the consecutive pin centres, producing them each way a little more than a panel length, so as to facilitate the erection of perpendiculars thereto. Then at each of the different centres erect a perpendicular to each centre line meeting there, and bisect the angle between the perpendiculars: the line of bisection will be the centre line of the post. Great care must be exercised in turning these right angles with the beam compasses, two points on each of the perpendiculars being found, so that if these two points and the centre be in exact line, the perpendicular may be relied on as correct. On each of these centre lines lay off the depth of the truss, and complete the skeleton diagram.

A partial check on the accuracy of the construction may be had by measuring the panel length of the top chord, which should agree with the length calculated as follows. Let

p = the increase in the panel length of the top chord above that of the bottom chord,

c = the camber at the centre of the span,

s = length of span,

d = depth of truss,

and

n = number of panels.

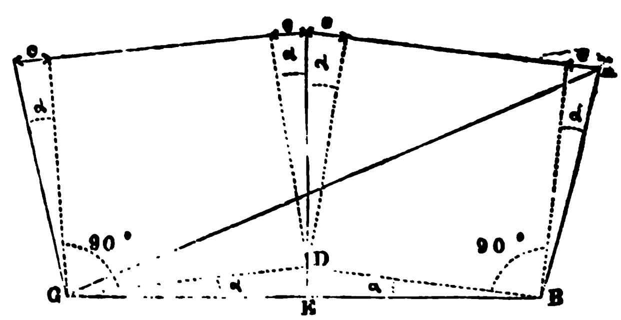
Then, according to the method given in Trautwine's "Pocket-Book," $p = \frac{8dc}{sn}$, where d and s may be measured in feet, and c and p in inches. The panel length of the top chord will then be $l' = l + p$, where l is the panel length of the bottom chord.

This is not a certain proof of the accuracy of the work. Two consecutive post centre lines might be equally inclined from their correct positions, and on the same side, though this would be shown in the next panel. A certain check must be obtained by measuring the lengths of the diagonals, which should be equal to each other, and agree with that found by the formula

$$D = \sqrt{d^2 + \left(l + \frac{p}{2}\right)^2},$$

where D is the length required.

In double intersection bridges the exact lengths of the long diagonals can be found by the following method.



Let l = panel length in bottom chord $= GD = DB$.

$c = \frac{1}{2}$ increase of panel length in top chord.

d = depth of truss between centres of chord $= AB$.

α = angle between radial line at panel pt. and $\perp$ to chord.

Then $\alpha = \sin^{-1}\frac{c}{d}$ (Eq. 1) $\quad DE : c :: l : d \quad \therefore DE = \frac{cl}{d}$

$$BG = 2\,GE = 2\sqrt{l^2 - \frac{c^2 l^2}{d^2}} = 2l\sqrt{\frac{d^2 - c^2}{d^2}} \quad \text{.................................(Eq.2)}$$

BG can be taken equal to $GD + DB = 2GD$ without perceptibly affecting the final result.

In triangle ABG, AB and BG are known; also angle $ABG = 90° + 2\alpha$.

$AB + BG : AB - BG :: \tan\frac{1}{2}\,[180° - (90° + \alpha)] : \tan\frac{1}{2}\,[BAG - BGA]$;

$$\therefore \qquad BAG - BGA = 2\tan^{-1}\left[\frac{AB - BG}{AB + BG}\tan\left(45° - \frac{\alpha}{2}\right)\right]$$

Again:

$$(BAG - BGA) + (BAG + BGA) = 2\,BAG =$$
$$(BAG - BGA) + (90° - 2\alpha), \text{ which gives } BAG;$$

also

$$BGA = 180° - (BAG + 90° + 2\alpha) = 90° - (BAG + 2\alpha):$$

finally,

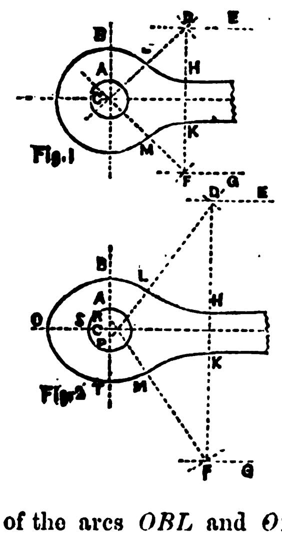
$$AG = AB\cos BAG + BG\cos BGA = \text{length of diagonal required.}$$

The length of the diagonal as manufactured should be one sixty-fourth of an inch less than that calculated, so as to allow for the play of the pins in the eyes. As a slightly greater allowance for play is permissible, it is better to take the next smallest sixty-fourth, if, after making the reduction just indicated, the length should not come out on an exact sixty-fourth. Because of the play in the pin holes of the bottom chord bars, the panel length of the manufactured top chord should exceed the calculated length by about a thirty-second of an inch.

Next fill out the elevation of the chords, posts, diagonals, and batter brace, without showing details. Alongside of each tension member show the heads with their dimensions, and on the shortened distance mark the size of the bars, the number of them, and the length from centre, to centre, as shown on Plate X.

At the right-hand end of the drawing, or on a separate sheet, whichever be more convenient, draw out the heads full size, placing one on the other, if this can be done without confusion. Sometimes as many as six heads can be represented about one centre, provided that both pins and heads diminish together.

For hammered heads the method of construction is very simple. It consists in describing, as in Fig. I of the accompanying diagrams, a circle of radius CA, equal to that of the pin hole, and a portion of another circle with a radius CB, equal



to that of the pin hole plus the product of one-half the depth of the bar HK by the ratio given in the table in Chapter VI., then drawing the lines DE and FG parallel to the sides of the bar, and at a distance therefrom equal to CB; and with C as a centre and a radius CD, equal to twice CB, describing an arc to intersect DE and FG in the points D and F; finally, with D and F as centres, and radii equal to CB, describing the arcs HL and KM, tangent to the sides of the bar at H and K, and to the outer circle at L and M.

For welded heads the construction is as shown in Fig. 2, where the pin hole and bar are laid out as before. The distance AB is equal to one-half of HK multiplied by the ratio given in the table in Chapter VI., and the distance SO is equal to HK, or the diameter of the pin hole, whichever be the greater. The centres P and R of the arcs OBL and OTM respectively are found by trial; then DE and FG are drawn parallel to the sides of the bar at distances therefrom DH and FK, equal to one and seven-tenths time PB or RT; and with P and R as centres, and radii equal to two and seven-tenths times PB or RT, or, what is the same thing, equal to DH plus PB, arcs are described cutting DE in D, and FG in F; finally, with D and F as centres, and with radii equal to DH, arcs are drawn tangent to the sides of the bar at H and K, and to the arcs OBL and OTM at L and M respectively.

These constructions, with slight modifications, are taken from Trautwine's "Pocket-Book."

Next show the posts and the attached sway bracing in two projections with all their details. There should be allowed a clearance of about an eighth of an inch for the ends of the posts inside of the chord. The positions for the stay plates should be as close to the pin as possible, allowing a little clearance for the diagonals. The proper positions can be ascertained from the general elevation. The lattice bars should be close to the stay plates: it will not be necessary to show more than a few of them on each strut, the positions of the others being indicated by their centre lines, as shown on Plate X.

Next show in two projections the top chord and batter brace with all their details, and give several views of each connecting-plate and other detail in the neighborhood of its position on the elevation. The joints in the channels and plate of the top chord should be located three or four inches to that side of each panel point which is farthest from the centre of the bridge, so that the pin holes shall be bored through a single piece, and through the thicker of the two abutting pieces. At the hip joint it is of course unavoidable to bore the pin hole through the abutting ends of the chord and batter-brace channels. Next pass to the plan, where the first thing to do is to draw parallel to the original horizontal line of the elevation traces of the central vertical planes of the trusses and of the central plane of the bridge, locating the panel points very carefully, and as nearly as possible vertically, below.

their corresponding positions on the elevation. Then arrange the chord packing on one side of the plan so as to make the bending-moments on the pins as small as possible without having any of the chord bars pull at too great an angle with the plane of the truss.

If any of the panels have trussed bars, the trussing should be here shown, and the spacing of the rivet holes for same in the chord bars should be represented close to the plan of the trussed bars. If there be a bottom chord strut, it should be represented in elevation alongside the plan of the packing.

Near the latter should be drawn separate views of the lower chord pins; giving their number, diameter, lengths between shoulders, diameters and lengths of reduced ends, and the total lengths; also the sizes of the nuts. At the right hand end of the drawing show the floor system proper for one panel length in plan and elevation, also a floor beam and a track-stringer each in two or three views, with the shims and their attachment to the stringers. Generally the floor beams will be all alike and symmetrical about the central longitudinal vertical plane of the bridge, so it will be sufficient to represent half a beam. The bearing for an end stringer should be completely delineated.

On one half of the plan show the lower lateral struts and rods attached to the lower chord and another view of each alongside, not forgetting to properly represent the turnbuckles; and on the other half of the plan show the upper lateral struts and rods and the portal bracing each in two views with all the details near by. Half lengths are generally sufficient for these struts and rods, but if full lengths be required they can be represented by moving the panel points of the lower half of the plan half a panel length to the right.

Plate X. was prepared by the above method, but was afterwards traced from the original sheet into a more compact form. The scale of $\frac{3}{4}'' = 1'$ used is smaller than should be employed in practice. This is both for economy of space and because the drawings were prepared for a model of the bridge designed in the last chapter. For the latter reason there are no details drawn on an enlarged scale. It was the author's intention to illustrate on this and succeeding plates all the working drawings for the model of the bridge, but he has been prevented from so doing by the unusually large estimate for printing this memoir. The complete drawings can be seen at any time in the rooms of the Civil Engineering Department of the Tokio Daigaku.

In writing dimensions, etc, upon a working-drawing, it is immaterial from which direction the writing be read; that is, it may be read sidewise, upside down, or in any direction most convenient to the draughtsman. In making tracings, this matter can be rectified if it be thought advisable. Full directions for the manufacturer should be written on the drawing.

Finally, take the list of members, and go carefully over the drawing with it; seeing not only that each piece is represented, but that there are sufficient measurements given to have it manufactured.

The following additional directions and hints may be found useful. Refer each group of rivets to some local line, which is itself referred to the end of the piece, or

some other prominent part. Show a section of each member, and write the dimensions of all channels, angles, I-beams, etc., near the section. Write along each piece its extreme length or lengths, its length from centre to centre of eyes, and of what it is composed. The ends of the two pieces of an adjustable rod should be separated by at least three or four inches in the turn buckle or sleeve nut. Mark what rivets are countersunk, and at which end. If the scale of the drawing be large enough, the rivetting can be thus represented — draw full parallel lines across the rivet for countersinking on the upper side, dotted parallel lines for countersinking on the lower side, two sets of parallel lines crossing each other at right angles for countersinking on both sides, and solid black circles for rivet holes to be left open. Be careful to always note how many pieces are to be made as shown and how many opposite hand, when there are both rights and lefts.

Lay out all bevelled ends on an enlarged scale say from half to full size, and mark their dimensions along the edges, referring all measurements to a transverse line through some well defined point as the centre of the pin hole. These measurements should be checked by calculation. The slight bevels at the joints of the top chord should be treated with as much accuracy as the bevels at the hip joints; but, as the bevel is very slight, it will be legitimate to put it all on one of the abutting ends making the other a square cut.

The centre lines for lacing bars on the under side of a strut should be dotted. In laying out a long row of rivets, for instance lattice rivets or those for the top plate of a chord or batter brace, calculate the distances of some of the intermediate rivet holes from one end of the strut, then interpolate the other holes; because, if the spacing be laid out continuously from one end with the dividers, any error in the span of the dividers will be multiplied by the number of times that the distance is laid off.

After laying out a complete system of rivets for any member, check by seeing that the sum of the distances between rivet holes plus the distance of each end rivet from the end of the member is equal to the total length of the member.

Make duplicates of as many parts of the bridge as possible even at the expense of a small amount of iron, not only to save time in draughting but also in the shop and to facilitate the work in erection. Arrange to have as few loose pieces for shipment as possible, and mark on the drawing of each connecting piece to what it is to be attached or whether it is to be left loose. Thus the hip connecting plates should be attached to the chords and braces and those for the top chord to that portion through which the pin hole is bored. If there be any reason to fear rough handling of the iron in transit, it may be necessary to send some of the connecting plates separately, but the more loose pieces the more field rivetting, and the more field rivetting, the greater the erecting expenses and the longer the time and the greater the risk in raising the bridge.

Rivet spacing should be as regular as circumstances will permit, and all changes in spacing should be made suddenly instead of gradually so as to facilitate the punching of the holes by machine.

All measurements should be in feet, inches and the following vulgar fractions

of inches viz., halves, quarters, eighths, sixteenths, thirty seconds and sixty fourths: workmen do not seem to understand decimals, so it is better not to use them. Avoid also the use of the development method, as it is beyond the comprehension of ordinary workmen. The lengths of all main members should be measured on the drawing then checked by calculation.

When nuts are placed in a confined position, for instance pin nuts in jaws, care should be taken that there be ample room for them to turn in, as it is very awkward and sometimes impossible to screw up a nut, which is stationary, by turning the pin; nuts in confined positions may be turned by hammering them eccentrically.

Be careful to design no connection in such a manner that there will be rivets that cannot be driven without inconvenience. This remark is especially applicable to field rivetting.

It must be borne in mind that, no matter how carefully the bill of iron was prepared, there will be many minor changes found necessary in making the working drawings; but, as a rule, such changes do not materially affect the total weight of iron in the bridge.

The following tables taken from Carnegie's "Pocket Companion" will be found very useful in making working drawings, as well as in preparing bills of iron.

UPSET SCREW ENDS FOR ROUND & SQUARE BARS.

Standard Proportions of the Keystone Bridge Company.

Dia. of Round or Side of Square Bar. Inches.	ROUND BARS.				SQUARE BARS.			
	Dia. of Upset Screw End. Inches.	Dia. of Screw at Root of Thread. Inches.	Threads Per Inch. No.	Excess of Effective Area of Screw End over Bar. Per Cent.	D'a. of Upset Screw End. Inches.	Dia. of Screw at Root of Thread. Inches.	Threads Per Inch. No.	Excess of Effective Area of Screw End over Bar. Per Cent.
1/2	3/4	.620	10	54	3/4	.620	10	21
9/16	3/4	.620	10	21	7/8	.731	9	33
5/8	7/8	.731	9	37	1	.837	8	41
11/16	1	.837	8	48	1	.837	8	17
3/4	1	.837	8	25	1 1/8	.940	7	23
13/16	1 1/8	.940	7	34	1 1/4	1.065	7	35
7/8	1 1/4	1.065	7	48	1 3/8	1.160	6	38
15/16	1 1/4	1.065	7	29	1 3/8	1 160	6	20
1	1 3/8	1.160	6	35	1 1/2	1.284	6	29
1 1/16	1 3/8	1.160	6	19	1 5/8	1.389	5 1/2	34
1 1/8	1 1/2	1.284	6	30	1 5/8	1.389	5 1/2	20
1 3/16	1 1/2	1.284	6	17	1 3/4	1.490	5	24
1 1/4	1 5/8	1.389	5 1/2	23	1 7/8	1.615	5	31
1 5/16	1 3/4	1.490	5	29	1 7/8	1.615	5	19
1 3/8	1 3/4	1.490	5	18	2	1.712	4 1/2	22
1 7/16	1 7/8	1.615	5	26	2 1/8	1.837	4 1/2	28
1 1/2	2	1.712	4 1/2	30	2 1/8	1.837	4 1/2	18
1 9/16	2	1.712	4 1/2	20	2 1/4	1.962	4 1/2	24
1 5/8	2 1/8	1.837	4 1/2	28	2 3/8	2.087	4 1/2	30
1 11/16	2 1/8	1.837	4 1/2	18	2 3/8	2.087	4 1/2	20
1 3/4	2 1/4	1.962	4 1/2	26	2 1/2	2.175	4	21
1 13/16	2 1/4	1.962	4 1/2	17	2 5/8	2.300	4	26
1 7/8	2 3/8	2.087	4 1/2	24	2 5/8	2.300	4	18
1 15/16	2 1/2	2.175	4	26	2 3/4	2.425	4	23
2	2 1/2	2.175	4	18	2 7/8	2.550	4	28
2 1/16	2 5/8	2.300	4	24	2 7/8	2.550	4	20
2 1/8	2 5/8	2.300	4	17	3	2.629	3 1/2	20
2 3/16	2 3/4	2.425	4	23	3 1/8	2.754	3 1/2	24
2 1/4	2 7/8	2.550	4	28	3 1/8	2.754	3 1/2	18
2 5/16	2 7/8	2.550	4	22	3 1/4	2.879	3 1/2	22
2 3/8	3	2.629	3 1/2	23	3 3/8	3.004	3 1/2	26
2 7/16	3 1/8	2.754	3 1/2	28	3 3/8	3.004	3 1/2	19
2 1/2	3 1/8	2.754	3 1/2	21	3 1/2	3.100	3 1/4	21
2 9/16	3 1/4	2.879	3 1/2	26	3 5/8	3.225	3 1/4	24
2 5/8	3 1/4	2.879	3 1/2	20	3 5/8	3.225	3 1/4	19
2 11/16	3 3/8	3.004	3 1/2	25	3 3/4	3.317	3	20
2 3/4	3 3/8	3.004	3 1/2	19	3 7/8	3.442	3	23
2 13/16	3 1/2	3.100	3 1/4	22	3 7/8	3.442	3	18
2 7/8	3 5/8	3.225	3 1/4	26	4	3.567	3	21
2 15/16	3 5/8	3.225	3 1/4	21	4 1/8	3.692	3	24
3	3 3/4	3.317	3	22	4 1/8	3.692	3	19
3 1/8	3 7/8	3.442	3	21	4 1/4	3.923	2 7/8	24
3 1/4	4	3.567	3	20	4 1/2	4.028	2 3/4	21
3 3/8	4 1/8	3.692	3	20	4 5/8	4.153	2 3/4	19
3 1/2	4 1/4	3.798	2 7/8	18				
3 5/8	4 1/2	4.028	2 3/4	23				
3 3/4	4 5/8	4.153	2 3/4	23				
3 7/8	4 3/4	4.255	2 5/8	21				

REMARKS. — As upsetting reduces the strength of iron, bars having the same diameter at root of thread as that of the bar, invariably break in the screw end, when tested to destruction, without developing the full strength of the bar. It is therefore necessary to make up for this loss in strength by an excess of metal in the upset screw ends over that in the bar.

The above table is the result of numerous tests on finished bars made at the Keystone Bridge Company's Works in Pittsburgh, and gives proportions that will cause the bar to break in the body in preference to the upset end.

The screw threads in above table are the Franklin Institute standard.

To make one upset end for 5″ length of thread allow 6″ length of rod additional.

STANDARD SCREW THREADS, NUTS AND BOLT HEADS.

Recommended by the Franklin Institute.

SCREW THREADS.

Angle of Thread 60°. Flat at Top and Bottom= ⅛ of pitch

Dia. of Screw. Inches.	Dia. at Root of Thread. Inches.	Threads per Inch. No.
¼	.185	20
5/16	.240	18
⅜	.294	16
7/16	.344	14
½	.400	13
9/16	.454	12
⅝	.507	11
¾	.620	10
⅞	.731	9
1	.837	8
1⅛	.940	7
1¼	1.065	7
1⅜	1.160	6
1½	1.284	6
1⅝	1.389	5½
1¾	1.490	5
1⅞	1.615	5
2	1.712	4½
2¼	1.962	4½
2½	2.175	4
2¾	2.425	4
3	2.629	3½
3¼	2.879	3½
3½	3.100	3¼
3¾	3.317	3
4	3.567	3
4¼	3.798	2⅞
4½	4.028	2¾
4¾	4.255	2⅝
5	4.480	2½
5¼	4.730	2½
5½	5.053	2⅜
5¾	5.203	2⅜
6	5.423	2¼

Nuts and Bolt Heads

are determined by the following rules, which apply to Square and Hexagon Nuts both:

Short diameter of rough nut
=1½×dia. of bolt +⅛in.

Short diameter of finished nut
=1½×dia. of bolt+1-16 in.

Thickness of rough nut.
=diameter of bolt.

Thickness of finished nut
=diameter of bolt—1-16 in.

Short diameter of rough head
=1½×dia. of bolt+⅛ in.

Short dia. of finished head
=1½×dia. of bolt+1-16 in.

Thickness of rough head
=½ short dia. of head.

Thickness of finished head
=dia. of bolt—1-16 in.

The long diameter of a hexagon nut may be obtained by multiplying the short diameter by 1.155, and the long diameter of a square nut by multiplying the short diameter by 1.414.

The above standards for screw threads, nuts and bolt heads, were recommended by the Franklin Institute in Dec. 1864. The standard for screw threads has been very generally adopted in the United States, but proportions recommended for nuts and bolt heads have not found general acceptance because of the odd sizes of bar—not usually rolled by the mills—required to make the nut.

WROUGHT SPIKES.

Number to a keg of 150 lbs.

Length. In.	¼ in. No.	5/16 in. No.	⅜ in. No.	Length. In.	¼ in. No.	5/16 in. No.	⅜ in. No.	7/16 in. No.	½ in. No.
3	2250	....	....	7	1161	662	482	445	306
3 1-2	1890	1208	....	8	....	635	455	384	256
4	1650	1135	....	9	....	573	424	300	240
4 1-2	1464	1064	....	10	....	....	391	270	222
5	1380	930	742	11	....	....	....	249	203
6	1292	868	570	12	....	....	....	236	180

SIZES AND WEIGHTS OF HOT PRESSED SQUARE NUTS.

The sizes are the usual manufacturers', not the Franklin Institute Standard. Both weights and sizes are for the unfinished Nut. The weights are calculated one cubic foot weighing 480 lbs.

Size of Bolt.	Weight of 100 Nuts.	Rough Hole.	Thickness of Nut.	Side of Square.	Diagonal.	No. of Nuts in 100 lbs.
1/4	1.5	7/32	1/4	1/2	.71	6800
5/16	2.9	9/32	5/16	5/8	.88	3480
3/8	4.9	11/32	3/8	3/4	1.06	2050
7/16	7.7	13/32	7/16	7/8	1.24	1290
1/2	8.6	7/16	1/2	7/8	1.24	1170
1/2	11.8	7/16	1/2	1	1.41	850
9/16	16.7	1/2	9/16	1 1/8	1.59	600
5/8	17.7	9/16	5/8	1 1/8	1.59	570
5/8	22.8	9/16	5/8	1 1/4	1.77	440
3/4	32.3	21/32	3/4	1 3/8	1.94	310
3/4	39.8	21/32	3/4	1 1/2	2.12	251
7/8	53.	25/32	7/8	1 5/8	2.30	190
7/8	63.	25/32	7/8	1 3/4	2·47	159
1	68.	7/8	1	1 3/4	2.47	146
1	94.	7/8	1	2	2.83	106
1 1/8	103.	15/16	1 1/8	2	2.83	97
1 1/8	137.	15/16	1 1/8	2 1/4	3.18	73
1 1/4	145.	1 1/16	1 1/4	2 1/4	3.18	69
1 1/4	186.	1 1/16	1 1/4	2 1/2	3.54	54
1 3/8	247.	1 3/16	1 3/8	2 3/4	3.89	41
1 1/2	319.	1 5/16	1 1/2	3	4 24	31.3
1 5/8	400.	1 7/16	1 5/8	3 1/4	4.60	24.8
1 3/4	500,	1 9/16	1 3/4	3 1/2	4.95	19.9
1 7/8	620.	1 11/16	1 7/8	3 3/4	5.30	16.2
2	750.	1 13/16	2	4	5.66	13.4
2 1/8	780.	1 7/8	2 1/8	4	5.66	12.8
2 1/4	930.	2	2 1/4	4 1/4	6.01	10.7
2 3/8	960.	2 1/8	2 3/8	4 1/4	6.01	10.4
2 1/2	1130.	2 1/4	2 1/2	4 1/2	6.36	8.9
2 3/4	1370.	2 7/16	2 3/4	4 3/4	6.72	7.3
3	1610.	2 11/16	3	5	7.07	6.2
3 1/4	2110.	2 15/16	3 1/4	5 1/2	7.78	4.7
3 1/2	2750.	3 1/8	3 1/2	6	8.49	3.6

SIZES AND WEIGHTS OF HOT PRESSED HEXAGON NUTS.

The sizes are the usual manufacturers', not the Franklin Institute Standard. Both weights and sizes are for the unfinished Nut. The weights are calculated, one cubic foot weighing 480 lbs.

Size of Bolt.	Weight of 100 Nuts.	Rough Hole.	Thickness of Nut.	Short Diameter.	Long Diameter.	No. of Nuts in 100 lbs.
1/4	1.3	7/32	1/4	1/2	.58	8000
5/16	2.4	9/32	5/16	5/8	.72	4170
3/8	4.1	11/32	3/8	3/4	.87	2410
7/16	6.8	13/32	7/16	7/8	1.01	1460
1/2	7.1	7/16	1/2	7/8	1.01	1410
1/2	9.8	7/16	1/2	1	1.15	1020
9/16	14.0	1/2	9/16	1 1/8	1.30	710
5/8	14.7	9/16	5/8	1 1/8	1.30	680
5/8	19.1	9/16	5/8	1 1/4	1.44	520
5/8	22.9	9/16	3/4	1 1/4	1.44	440
3/4	27.2	21/32	3/4	1 3/8	1.59	370
3/4	39.	21/32	7/8	1 1/2	1.73	256
7/8	44.	25/32	7/8	1 5/8	1.88	226
7/8	50.	25/32	1	1 5/8	1.88	198
1	57.	7/8	1	1 3/4	2.02	176
1	64.	7/8	1 1/8	1 3/4	2.02	156
1 1/8	96.	15/16	1 1/4	2	2.31	104
1 1/4	134.	1 1/16	1 3/8	2 1/4	2.60	75
1 3/8	180.	1 3/16	1 1/2	2 1/2	2.89	56
1 1/2	235.	1 5/16	1 5/8	2 3/4	3.18	42
1 5/8	300.	1 7/16	1 3/4	3	3.46	33.4
1 3/4	370.	1 9/16	1 7/8	3 1/4	3.75	26.7
1 7/8	460.	1 11/16	2	3 1/2	4.04	21.5
2	450.	1 13/16	2	3 1/2	4.04	22.4
2 1/8	560.	1 7/8	2 1/8	3 3/4	4.33	18.0
2 1/4	560.	2	2 1/4	3 3/4	4.33	17.7
2 3/8	680.	2 1/8	2 3/8	4	4.62	14.7
2 1/2	810.	2 1/4	2 1/2	4 1/4	4.91	12.3
2 3/4	980.	2 7/16	2 3/4	4 1/2	5.20	10.2
3	1150.	2 11/16	3	4 3/4	5.48	8.7
3 1/4	1340.	2 15/16	3 1/4	5	5.77	7.5
3 1/2	1580.	3 1/8	3 1/2	5 1/4	6.05	6.3

DECIMALS OF AN INCH FOR EACH, $\frac{1}{64}$th.

$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction	$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction
	1	.015625			33	.515625	
1	2	.03125		17	34	.53125	
	3	.046875			35	.546875	
2	4	.0625	1–16	18	36	.5625	9–16
	5	.078125			37	.578125	
3	6	.09375		19	38	.59375	
	7	.109375			39	.609375	
4	8	.125	1–8	20	40	.625	5–8
	9	.140625			41	.640625	
5	10	.15625		21	42	.65625	
	11	.171875			43	.671875	
6	12	.1875	3–16	22	44	.6875	11–16
	13	.203125			45	.703125	
7	14	.21875		23	46	.71875	
	15	.234375			47	.734375	
8	16	.25	1–4	24	48	.75	3–4
	17	.265625			49	.765625	
9	18	.28125		25	50	.78125	
	19	.296875			51	.796875	
10	20	.3125	5–16	26	52	.8125	13–16
	21	.328125			53	.828125	
11	22	.34375		27	54	.84375	
	23	.359375			55	.859375	
12	24	.375	3–8	28	56	.875	7–8
	25	.390625			57	.890625	
13	26	.40625		29	58	.90625	
	27	.421875			59	.921875	
14	28	.4375	7–16	30	60	.9375	15–16
	29	.453125			61	.953125	
15	30	.46875		31	62	.96875	
	31	.484375			63	.984375	
16	32	.5	1–2	32	64	1.	1

The following extract on "Workshop Drawings" from the *American Engineer* of Nov. 7th the and 21st 1884, though perhaps not in exact accordance with the previous part of this chapter, contains so many useful hints that it has been considered advisable to insert it.

"The following article is by Alfred D. Ottewell, who can speak with authority from a life long experience in the best shops of Europe and America:

While each draughtsman has his own method, and each firm its own rules, it is believed that the following process of preparing working drawings, having given the general design and dimensions of the article to be manufactured, may contain points worthy of the consideration and adoption of both; being the result of an extensive practice and detailed observation.

PROCESS.

I. Draw large scale details of general connections—*i. e.*, where the separate pieces are joined together. These details are made chiefly for office use, though parts of them may be traced for the shops. Their principal use is to enable the draughtsman to draw out each part separately for the shops. These details need not be inked in or traced, except for purposes stated in III, as the subsequent drawings are complete in themselves for workshop use or reference.

II. Draw each member separately in pencil by the previous details made. If the draughtsman traces his own drawing the dimension need not be put on this pencil drawing, except guiding dimensions for reference. It is easier and saves time to put the dimensions directly on the ink tracing, since the tracing is more readable than the pencil drawing; whilst it also saves the time necessary to trace the dimensions.

III. Trace pencil drawing spoken of in II on tracing cloth. If any parts of the details in drawing mentioned in I are necessary to explain any part of the members shown on this drawing it is traced in juxtaposition with those members. By this means all the details of one member are shown together; if possible, on one drawing; thereby preventing confusion of workmen, inspectors, draughtsmen, and everyone connected with the work.

The process adopted by the author of making this tracing is as follows:

i. Tracing in circles or parts of circles.
ii. ,, ,, lines showing straight parts of members.
iii. ,, ,, freehand curves of members.
iv. ,, ,, centre lines.
v. Putting in dimension lines.
vi. Marking off dimension points or arrow heads.
vii. Writing dimensions.
viii. Sectioning parts shown in section.
ix. Giving a separate mark to each piece.
x. Putting on bill of material shown on drawing.
xi. Writing title, including name of job and order number.
xii. Entering drawing in "Drawing Book" and putting number on drawing. Signature of draughtsman and date.

i Preceeds ii, since it is easier to trace two straight lines to touch a circle than it is to trace a circle to touch two straight lines.

iii. Succeeds i and ii for a similar reason.

iv. Is in the position shown to adjust for any inequalities in tracing; such as tracing a circle slightly out of centre on drawing.

vi. Preceeds vii. since it facilitates dispatch, as the questions "What dimensions will best suit the requirements of the workman?" and "What size those dimensions actually are?" are considered separately instead of alternately, as is generally the case.

viii. Succeeds vii, as it often happens that sectioning and dimensions should cover the same ground, in which case the former can be left out, thereby making the dimension much more readable.

The bill of material x, gives the number and size to order, of articles required to be furnished to satisfy that drawing, and is made as follows, for example :

MATERIAL FOR TWO CRANES.

No.	DESCRIPTION.	LENGTH.	MARK.
2	Forgings	....	AA.
6	Angles 3" × 3" × ½"	6' 3½"	AB.
4	Plates 12" × ⅜"	7' 6½"	AC.
12	¾" Bolts. H. H. and N.	6"	AD.
16	½" Tap Bolts. H. H.	1½"	AE.
4	½" Bolts. CK. H. SQ. N.	2"	AF.

The marks AA, AB, etc., are shown on the details to which they refer.

ix Is adopted to give each piece a mark to distinguish it readily from other articles, and also to assist the formation of x, as the bill x is a better arrangement than the old plan of marking off on each piece the number required, as it shows the workman readily all that is on the drawing for him to make, and enables him to see at once when he is through with the work shown on the drawing.

The title, xi, should be in the same relative position on all drawings ; preferably the bottom right hand corner facing the drawing, as this is the most convenient for reference when filed away in drawers or rolls. Whether the tittle should or should not give the name of the firm or work for which it is for is an open question. The advocates for the affirmative say it is easier to remember a name, because more interesting, than an order number ; whilst the advocates for the negative say such information makes the work too public in many cases.

xii, The signature, preferably initials, of the draughtsman should be given on the drawing to facilitate reference to him should any question arise.

After the tracing has been signed by the draughtsman let it be,

IV. Examined or checked by another draughtsman. In this examination every dimension and note on the tracing is to be examined ; and all the necessary dimensions and information for the workman shown to be on the tracing ; and the fact ascertained that no two parts of the work will foul with each other.

When the examiner has satisfied himself on these points, his signature on the tracing is required to attest the fact, which signature is a guarantee to the foreman that the tracing is ready to work to.

Sometimes the tracing is made and examined by one and the same draughtsman. This is a quicker plan than the former, as the draughtsman is more familiar with the work on his own drawing, but it is not as safe, as the draughtsman is liable to make the same mistake twice over.

After the tracing has been signed by the examiner let a

V. Blue print be taken from the tracing and sent into the shops to the foreman engaged on the work, or to the "work's manager," as the case may require. The

name of blue print should be entered in a book with the name of foreman sent to, and the date on which it is sent. These particulars are useful to refer to, in case of an investigation being made into any delay in the proceedure of the work.

If the blue print is taken directly from the drawing made on thin, semi-transparent drawing paper for the purpose, the reader will readily see the slight modifications to be made to the preceeding remarks.*

The foregoing process may appear at first sight a lengthy one, but as it has been formed with the primary objects of securing great accuracy and dispatch in manufacture, it is believed that slight excess, if any, in cost of drawings made by this plan is doubly repaid by the saving in cost of manufacture. If, however, two drawings giving the same information and of equal efficiency are produced, one made by the above process and one made by any other process, it is believed the former will be made in less time, and with less labor, than the latter. The principle of doing one thing at a time is adopted throughout, and the practice sometimes adopted of finishing and tracing one part of a drawing before another part is commenced is, in this light, obviously wrong, unless there are especial reasons for doing so.

II.—RULES TO BE OBSERVED IN THEIR PREPARATION.

I. Put as little as possible on drawings, to prevent confusion, but everything necessary for the construction of the articles represented.

Unnecessary details, full or dotted lines, circles, shading, dimensions, notes, etc., involve a waste of time for both draughtman and workman, besides making the drawing confusing or more unreadable. Everything on the drawing should have a definite right to be there, so that if dimensionse tc., are repeated, it should be to facilitate the workman in understanding the drawing or in avoiding a liability to misunderstand. Often draughtsmen, either from want of thought or because it is easy and pleasant occupation, draw in detail, rivets, bolts, etc., where centre lines would do equally as well. If the drawings were to be pictures intended to be understood by the untrained mind, such details would be necessary, but such is not the case; working drawings are made up of a series of symbols which are understood to represent certain articles, materials or methods, and a workman will often understand a centre line, with given dimensions, to represent a rivet or bolt where details would tend to make the drawing confusing.

II. Make dimensions to suit the workman's requirements in preference to your own.

The draughtsman should, to carry out this rule, mentally go through the process of making the article represented on the drawing just as the workman will make it. It wastes a workman's time to require him to add and subtract a number

* Such paper can be obtained from Charles Schleicher and Schüll, Düren, Rhenish Prussia. It is numbered 785, and comes in rolls 33 yds. long, by 38 to 40 inches wide. Price per roll 25 shillings.

By its use considerable time will be saved in making blue prints.

J. A. L. W.

of unnecessary dimensions to find an essential one, besides making him liable to err. The draughtsman can more easily find such a dimension himself, and when once written on the drawing it is there for all the workmen requiring it.

III. Make as few notes as possible on a drawing; put dimensions or symbols instead.

Such notes are easily missed and are seldom read. They belong more to specifications than to drawings, and where instruction has to be given on drawings it can generally be given in a more readable form by symbols that catch the eye than by writing.

By "symbols" in this case is meant the usual marks adopted by different firms to indicate drilled holes, tapped holes, countersunk holes, holes left open, parts machined, steel, brass, etc. Such symbols are seen at a glance whilst a note requires more or less study, is more verbose and is more easily missed by the workman.

IV. Put all details and information required to make each article on one and the same drawing.

To give one detail of an article on one drawing, another on another, and so on, requires the workman making that article to monopolize those drawings and prevents other workmen from using them for other parts of the work. The article itself is made under greater difficulties, as it confuses the workman if he is required to pick out the details from several drawings and from among other details.

V. Put all dimensions for each article as near together as convenient.

It wastes a workman's time to require him to hunt for the dimensions of the piece he is making in the different parts of the same drawing. Thus if a web plate of a girder, say, be shown on the drawing the size may be given 8′ 6″ × ⅜″ × 7′ 3″ say, instead of showing the width in one place, thickness in another, and length in another.

VI. Make thick lines and thick plain block figures. These are more easily seen and read by the workman in the shops. The blue prints from such tracings are more uniform and distinct. Such lines and figures are not so soon obliterated or soiled by the dirt of the shops. The figures should be plain; the object being to make them easily readable, more useful than ornamental. Ornamental figures and lettering should be reserved for designs, estimates, etc.

VII. Give inspector's dimensions.

These are generally main dimensions. In examining the work, and in the work when in place, the intermediate dimensions are not of much importance. For example, the dimensions used by an inspector examining the columns for a building are, say, distance from underside of bearing plate to bearing surface for shoe of rafter or principal; distance from underside of bearing plate to centre line of holes for connecting floor beams to columns. The former dimension, being a main dimension, would generally be given as required, but the latter is often to be obtained only by additions and subtractions, trying to the patience of inspectors or examiners in inclement weather.

VIII. Instead of the term "right hand," use "as shown," and instead of "left hand," use "opposite hand."

The terms "right hand," "left hand" are ambiguous, since they do not say which is as drawn and is which opposite hand. For example, if there are twelve articles required for a given detail, nine of them to be right hand and the remaining three left hand, the order is ambiguous since it does not say whether the drawing represents a right hand article or a left hand article. If the order is nine required as shown, and three required opposite hand no such ambiguity exists.

IX. Let the drawings of cast-iron work be separate from the drawings of wrought iron work.

The pattern-maker should have no use for the wrought iron work, nor should the template maker or smith be required to hunt his work out of a mass of cast iron work.

X. Use rounded corners in castings where good, strong, durable castings are required.

Such rounded corners add strength, since the contraction on cooling is more even. They secure better castings, since the sand of the corners in the mould is not so liable to be washed away into any parts of the casting; they secure also more durable castings since the corners are not so liable to be knocked off by use. Of course the patterns with rounded corners for rough odd castings may be too expensive, unless the above advantages are more important than the cost.

XI. Put no scale on the drawings.

The workman is in no case to scale the drawing, except certain full size details, but should work to dimensions given. Scaling a drawing is obviously wrong. If the draughtsman has made the drawing to scale throughout, which is not always the case, the pattern maker, fitter and smith will often on scaling dimensions give different results, thus making it difficult to fit the different parts of the work together. If a required dimension is not given the draughtsman should be requested to give it. Putting a scale on a drawing is therefore worse than useless, since it tempts the workman to use it where dimensions have been accidentally omitted.

XII. Accuracy and completeness in execution are more valuable than dispatch.

This might almost be considered an axiom, but if based upon definite reasons it will have a firmer hold upon the mind of the draughtsman. An error on a drawing becomes more costly to make the longer the correction is delayed. Take the following example: On the drawing of a cast-iron steam engine cylinder the inside diameter is given 1′ 8″, but on the drawing of the piston the diameter is given 18″. *How* the mistake occurred is apart from the argument, since mistakes do occur for which no reason can be given. If the draughtsman sees the mistake as soon as made it will only cost a few moments of his time to correct it. If the examiner detect it, it will cost somewhat more. If the workman or foreman should happen to detect it before constructing the piston a little more expense is incurred in correcting the mistake; and if the piston and rings are cast, on detection much more expense is incurred , whilst if the piston is machined complete, ready for fitting, much more expense is incurred in correction. The argument is that the draughtsman should work with care, and not that he should go over the same ground twice, as it is the examiners duty to thus check the drawing; for it is obviously a waste of time for the examiner

to examine work that has been previously examined.

XIII. Make as few variations as possible.

In repetition there is economy. If two castings can be made from one new pattern they will cost less than if they require a new pattern made for each of them. Again, if the two castings are slightly different from one another, the pattern can probably be made to suit both with a slight alteration, in which case the two castings will cost less than if they are designed so that they will require two whole new patterns. A saving is effected if the two castings can be designed alike without inconvenience, instead of opposite hand to each other, as, even if each does not require a separate pattern, the time taken by the moulder in taking the pattern for alteration to the pattern maker, and the time taken by the pattern maker in altering the pattern is entirely lost. Very often castings can be designed by the careful draughtsman to be cast from one pattern slightly modified to suit each case, and by this means unnecessary expense saved.

Rivets of equal pitch cost less to mark off on template than if of unequal pitch, since the layer-off has to set his dividers once in the former case, and a number of times in the latter. Adopting the same sized rivet or bolt throughout a structure is often cheaper than varying the size to suit theoretical requirements, as the additional labor in changing punches, drills, cores, etc., the additional handling and liability to err, in the latter case are more costly than the waste of material in the former. This principle of the economy in repetition is of the greatest importance and cannot be too widely studied."

The next group includes the connecting plates at the hips and intermediate panel points. The widths of the inner plates are, of course, equal to the depths of the channels which they connect, and those of the outer ones as great as will permit of their lying between the heads of the rivets which pass through the flanges: in case of necessity some of these heads might be countersunk so as to permit of a greater depth of outer connecting plate.

The limiting thicknesses and lengths of these plates are given in the following table, which has been prepared under the assumption that Carnegie's sections are employed and that the rivets are spaced as closely as good practice will permit.

When applied to the hip connection, the lengths of these plates are equal to the sum of the distance from the centre of the pin hole to the end of the plate on the chord, and that from the same point to the end of the plate on the batter brace. It must be understood that very heavy shallow channels are not used, but that the most economic size for the whole truss is in every case employed.

Depth of [	Weight of [	Thickness of Plate	Total Length of Plate
8″	12.5# to 17#	$\frac{3}{8}$″	32″ to 44″
9″	14.5# to 21#	$\frac{3}{8}$″ to $\frac{7}{16}$″	32″ to 46″
10″	16# to 26#	$\frac{3}{8}$″ to $\frac{7}{16}$″	32″ to 48″
12″	20# to 45#	$\frac{3}{8}$″ to $\frac{3}{4}$″	36″ to 66″
15″	40# to 60#	$\frac{7}{16}$″ to $\frac{3}{4}$″	42″ to 70″

The next on the list are the reinforcing and connecting plates at the pin holes in bottom chord struts. The former may always be made $\frac{3}{8}$″ thick and three or four inches deeper at the middle than the strut channels. Their length should be about three feet. Where a joint occurs in the bottom chord strut, the thickness of each connecting plate should be $\frac{1}{2}$″, and the depth at the middle from four to six inches greater than that of the channel, the increase being directly proportional to the depth of the channel. A length of four feet will be generally about right.

Next on the list come the reinforcing or connecting plates at the shoes.

The thickness for these may be taken as $\frac{7}{16}$″ for eight and nine inch channels, $\frac{1}{2}$″ for ten and twelve inches channels, and $\frac{5}{8}$″ for fifteen inch channels. The height of each vertical portion may be taken equal to twice the depth of the batter brace channels, and the length of the horizontal portion about the same or little more.

Next come the reinforcing plates at the feet of the posts. If the channels be light, not greater in depth than twelve inches, and with flanges untrimmed, a thickness of $\frac{3}{8}$″ will be sufficient for each inner and outer plate. For heavy sections of any depth, for all fifteen inch channels, and for channels with their flanges trimmed away it will be well to increase the thickness to half an inch. The width of the outer plate should be made as great as will fit into the channels, and for small channels it is well to make the horizontal portions of these plates wider than the vertical portions, so as to give room for attaching to the floor beam. The width of the inner plates should be at least as great as the depth of the post channels. If the latter be

small, the horizontal part of the aw formed by the inner reinforcing plates may be made as wide as the upper flange of the floor beam. Both outer and inner plates should be long enough to extend nearly to the upper edges of the stayplates.

Next come the reinforcing plates at the middle of posts. These are placed preferably on the inside of the posts, but may be put on the outside if there be any special reason for so doing. When on the inside the thickness should be equal to that of the web of the channel, but never less than $\frac{3}{8}''$. The width should be equal to the depth of the post channel.

If placed upon the outside of the post, the thickness should be increased $\frac{1}{8}''$ to allow for the necessarily diminished width. The length should be about twice the depth of the channels, but should in all cases be great enough to extend two or three inches within the stay plates.

Next come the bent connecting plates for intermediate struts to posts. These may be made of $\frac{3}{8}'' \times 4''$ iron about thirty inches long.

The connecting plates for brackets to lower portal struts, and for name plates to upper portal struts may be made of the length and thickness used in Chapter XVIII: the width, of course, will depend upon the distance apart of the portal struts channels.

The dimensions of the plates on the side braces and for connecting the side braces to the floor beams may either be scaled from Plate IX., which is drawn to a scale of one twelfth, or may be left to the judgement of the draughtsman. The bent plate at the upper end of the side brace should be at least $\frac{3}{8}''$ thick.

The thickness of a connecting plate for track stringers over a floor beam, when the former abut against the latter, need be no greater than $\frac{3}{8}''$, and the same thickness will serve for the connecting plates for stringers to floor beams. Octagonal plates between stringers and beams, used when the former rest upon the latter, should be $\frac{3}{4}''$ thick.

The other dimensions for connecting members between stringers and beams can be scaled from the plates.

Cover plates should have a thickness equal to that of the plates, the joints of which they cover, and they should be a few inches longer than their width.

The dimensions of filling plates need no comment.

The sectional area of an extension plate should be twice as great as that of the channel to which it is attached, and its width should be equal to that of the channel. It is composed of two equal or nearly equal thickness, one of which should extend three or four inches below the upper edge of the stay plate, and the other at most a foot farther than the first.

The thickness of outer jaw plates for lateral struts having no vertical pins should be $\frac{3}{8}''$, the width at least equal to that of the channels and the length from three to three and a half feet. The inner jaw plate is necessarily shorter but usually of the same width: the thickness may be taken as $\frac{1}{2}''$.

For lateral struts with vertical pins passing through the jaw plates, the thickness of the latter will have to be from $\frac{5}{8}''$ to $\frac{3}{4}''$ and the width at the pin hole from 7″ to 9″. the length required will be from four to four and a half feet.

For portal struts the jaw plates will have to be from 1/2" to 5/8" thick and from three to four feet long. If there be no portal vibration rods, the width of plate may be made equal to the depth of the portal strut channels, otherwise it should be about four inches greater.

The thickness for any shoe or roller plate is given in Chapter VI. The width of shoe plate should be about four inches greater than that of the batter brace plate, and the length three inches or more longer than twice the depth of the batter brace channels.

The width of a roller plate should be about six inches greater than that of the shoe plate, and its length eight or nine inches greater than the length of the same.

Track stringer and plate girder bed plates should be about 3/4" thick and 14" square.

Beam hanger plates should be about 1 1/8" thick and large enough to provide full bearing for the hanger nuts.

The sizes for latticing and lacing bars for all cases are given in Tables XX. and XXI.

Trussing bars should vary in section from 1/2" × 8" to 5/8" × 8 1/2".

The sizes for all truss pins can be interpolated from the diagrams of stresses, and those for vertical pins attaching *single* lateral rods to jaws can be taken from the following table.

SIZE OF ROD OR BAR		DIAMETER OF IRON PIN
Round	Square	
1 13/16"	1 9/16"	3"
1 7/8"	1 5/8"	3 1/8"
1 15/16"	1 11/16"	3 1/8"
2"	1 3/4"	3 1/4"
2 1/16"	1 13/16"	3 1/4"
2 1/8"	1 7/8"	3 3/8"
2 3/16"	1 15/16"	3 3/8"
2 1/4"	2"	3 3/8"
2 5/16"	2 1/16"	3 1/2"
2 3/8"	2 1/8"	3 5/8"

It seems almost unnecessary to state that the diameter of the greater of the two lateral rods attached to the pin is to be used in determining the size of the latter.

When double lateral rods are employed, the sizes of vertical pins through the jaws can be obtained from the following table. The bending moments are so great that it will generally be found necessary to employ steel pins.

SIZE OF ROD OR BAR		DIA. OF STEEL PIN	DIA. OF IRON PIN
Round	Square		
1 13/16"	1 9/16"	3 1/4"	3 3/4"
1 7/8"	1 5/8"	3 3/8"	3 7/8"
1 15/16"	1 11/16"	3 3/8"	4"
2"	1 3/4"	3 1/2"	4 1/8"
2 1/16"	1 13/16"	3 5/8"	4 3/8"
2 1/8"	1 7/8"	3 3/4"	4 1/2"
2 3/16"	1 15/16"	3 7/8"	
2 1/4"	2"	4"	
2 5/16"	2 1/16"	4 1/8"	
2 3/8"	2 1/8"	4 1/8"	

Brackets for connecting intermediate struts to posts or lower portal struts to batter braces may be made of $2\frac{1}{2}'' \times 2\frac{1}{2}'' — 4.9^{\#}$ angle iron: those for connecting upper lateral struts to posts may vary from the same size for short panels and low trusses to that of $3\frac{1}{2}'' \times 3\frac{1}{2}'' — 8.3^{\#}$ angle irons for long panels and deep trusses. Those for connecting portal struts to batter braces when there are no portal vibration rods may vary from $3'' \times 3'' — 5.9^{\#}$ angles for 70′ span to $4'' \times 5'' — 22^{\#}$ angles for 150′ or 160′ spans. The larger these brackets the greater the number of rivets necessary for their attachment, and the greater the diameter of the rivets. In proportioning brackets it is well to err on the side of safety.

Brackets for connecting track stringers to floor beams may be made of 8″ or 9″ light channels, and those for connecting floor beams to top chords in deck bridges of 6″ or 7″ light channels.

The sizes of beam hangers are given in Table VII.

The diameters of rollers may vary from 2″ for short spans to 8″ for long ones, the spaces between them being about an inch and their length nearly equal to the width of the shoe plate.

Shoe pin supporting pieces should be made of heavy I-beam with thick web.

The anchor pieces for sides of roller plates should not be made less than $\frac{1}{2}''$ thick, and should be as long as the roller plates.

Stringer bracing frames may be made of $3'' \times 3'' — 7.2^{\#}$ angle iron: the same size may be advantageously employed for stiffening angles of track stringers and floor beams, though somewhat lighter sections may often be employed.

Stringer supporting shelves may be made of $4'' \times 6'' — 16^{\#}$ angle iron.

Splice plates for webs of track stringers should be $\frac{3}{8}''$ thick, as deep as possible, and wide enough for four rows of rivets: those for the flanges should have a sectional area a little greater than that of the flanges themselves and should be long enough to contain at least a dozen rivets on each side of the joint.

The rest of the details on the list of members have either been so fully treated elsewhere or are so simple in their construction that their sizes may be readily determined without any further explanation.

CHAPTER XXI.

ORDER BILLS AND SHIPPING-BILLS.

When there is necessity for haste in building a bridge, time can be saved by sending a partial order bill to the manufacturers before starting to make the working-drawings, or after they are partially pencilled.

Such preliminary order bills should include only those portions which are termed in this treatise "Main Members," and those details of the sizes of which the designer is certain; for instance, stay plates, pins, brackets, and the plates and angles for built beams.

The length of the main members in the bill should be three-quarters of an inch greater than will actually be required, in order to allow for the dressing of rough ends, and, should there be any doubt in the designer's mind concerning the exact length of any piece, he should make the ordered length great enough to cover any variation which there may be in the design.

Of course, where there are bevelled ends on a piece, the extreme length plus the allowance for waste must be given.

Where a number of small pieces are to be cut from one large piece, an extra allowance of length must be made to provide for the waste in cutting, say from an eighth to a quarter of an inch for each short length. After finishing the pencilling for a working-drawing, the remainder of the preliminary order bill may be made out and sent. It should be divided into the following groups, containing the measurements indicated:—

Channels	No.	Depth	Weight per foot	Length	Finished length

Angles	No.	Thickness	Legs	Weight per foot	Length	Finished length

I-beams	No.	Depth	Weight per foot	Length	Finished length

Plates	No.	Thickness	Width	Length	Finished length

Eye bars	No.	Thickness	Depth	Depths of heads	Length centre to centre of eyes	Extreme length

Adjustable rods with plain eyes	No.	Diameter.		Short Piece.			Long Piece.		
		Rod	Upset end	Diameter of eye	Length of loop	Length, centre of eye to end	Diameter of eye	Length of loop	Length, centre of eye to end

Adjustable rods with bent eyes	No.	Diameter.		Short Piece.		Long Piece.	
		Rod	Upset end	Diameter of eye	Length, centre of bend to end	Diameter of eye	Length, centre of bend to end

Pins	No.	Diameter.		Length between shoulders	Extreme length
		Body	Reduced ends		

Rollers	No.	Length between shoulders	Extreme length

Any details which will not go into one of these groups will be made of material that the manufacturer keeps in stock; for instance, fillers, washers, nuts, turn buckles, sleeve nuts, ornamental work, name plates &c.

Pins should be ordered an eighth of an inch greater in diameter than required in the bridge, so that they may be turned down, and shoe plates and roller plates one sixteenth of an inch thicker so as to allow for planing.

Spikes are generally purchased separately from special dealers.

Lumber is, of course, bought separately; it should be ordered in the following form.

No. Pieces	Thickness	Width	Length	Kind of Wood

After the working drawing is finished there should be prepared to accompany it a final order bill in which are to be grouped all similar pieces and all their details which are attached to them in the shop.

The following grouping will cover any case of an ordinary iron railroad bridge designed according to the method of this treatise.

TOP CHORD SECTIONS.

Channels	No.	Depth	Weight per foot	Finished length
I-beams...	No.	Depth	Weight per foot	Finished length
Angles	No.	Legs	Thickness	Finished length
Long-plates	No.	Width	Thickness	Finished length
Cover-plates	No.	Width	Thickness	Finished length
Stay-plates	No.	Width	Thickness	Finished length
Lattice-bars	No.	Width	Thickness	Finished length
Connecting-plates ...	No.	Width	Thickness	Finished length

BATTER BRACES.

Channels	No.	Depth	Weight per foot	Finished length
I-beams...	No.	Depth	Weight per foot	Finished length
Angles	No.	Legs	Thickness	Finished length
Long-plates	No.	Width	Thickness	Finished length
Cover-plates (Hip) ...	No.	Width	Thickness	Finished length
Stay-plates	No.	Width	Thickness	Finished length
Lattice-bars	No.	Width	Thickness	Finished length
Connecting-plates ...	No.	Width	Thickness	Finished length
Shoe-plates	No.	Width	Thickness	Finished length

POSTS.

Channels	No.	Depth	Weight per foot	Finished length
Angles	No.	Legs	Thickness	Finished length
Long-plates	No.	Width	Thickness	Finished length
Stay-plates	No.	Width	Thickness	Finished length
Lattice-bars	No.	Width	Thickness	Finished length
Extension-plates	No.	Width	Thickness	Finished length
Reinforcing-plates ...	No.	Width	Thickness	Finished length

BOTTOM CHORD STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Re-enforcing plates ...	No.	Width	Thickness	Finished length
Connecting plates ...	No.	Depth	Thickness	Finished length

UPPER LATERAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length

LOWER LATERAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
I-beams...	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length

PORTAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lattice bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length
Connecting - plates for brackets to channels.	No.	Width	Thickness	Finished length
Connecting - plates for name plates to channels.	No.	Width	Thickness	Finished length

INTERMEDIATE STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
I beams...	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing bars	No.	Width	Thickness	Finished length
Connecting plates to posts	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length
Connecting plates for brackets to channels.	No.	Width	Thickness	Finished length

STIFFENED HIP VERTICALS.

Channels	No.	Depth	Weight per foot	Finished length
Flat bars	No.	Width	Thickness	Length centre to centre of eyes, and extreme length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Re-enforcing plates ...	No.	Width	Thickness	Finished length
Trussing	No.	Width	Thickness	Finished length

MAIN DIAGONALS AND PLAIN CHORD BARS.

No.	Depth	Thickness	Depth of heads	Thickness of heads	Diameter of eyes	Length centre to centre of eyes	Extreme length

HIP VERTICALS AND COUNTERS.

No.	Section	Diameter of enlarged end	Lengths of loop eyes	Lengths centre of eyes to ends, or centre of eye to centre of eye

LATERAL AND VIBRATION RODS.

No.	Diam.	Diameter of enlarged end	Length centre of eye to bend or loop	Length centre of bend or eye to end of short piece	Length centre of bend or eye to end of long piece

STRUTS OF TRUSSED CHORD BARS.

No. of struts	Section	Sizes of heads	Section of trussing	Length of trussing	Length of strut centre to centre of eye	Extreme length of strut

SIDE BRACING.

No.	Section	Size of connecting-plates	Size of reinforcing plates	Extreme length of brace

FLOOR BEAMS.

Plates . .	No.	Width	Thickness	Finished length
Angles . .	No.	Legs	Weight per foot	Finished length
Tees . .	No.	Legs	Weight per foot	Finished length

TRACK STRINGERS.

I-beams . .	No.	Depth	Weight per foot	Finished length
Angles . .	No.	Legs	Weight per foot	Finished length
Plates . .	No.	Width	Thickness	Finished length

ROLLER AND BED PLATES.

Plates . .	No.	Width	Finished Thickness	Finished length
Angles . .	No.	Legs	Weight Per foot	Finished length

NAME PLATES.

No.	Date.

OTHER SEPARATE PLATES.

No.	Width	Thickness	Finished length

PINS AND THEIR NUTS.

No.	Diameter	Size of Nuts	Extreme length

BOLTS AND THEIR NUTS.

No.	Diameter	Size of nuts	Length under head, or extreme length

BRACKETS.

Angles	No.	Legs	Weight per foot	Extreme length
Channels ...	No.	Depth	Weight per foot	Extreme length
Tee-iron	No.	Legs	Weight per foot	Extreme length

ORNAMENTAL WORK.

No. of pieces	Description

GUARD RAILS.

No. of pieces	Section	Weight per foot	length

BEAM HANGERS AND THEIR NUTS.

No.	Section	Diameter of upset end	Diameter of eye	Size of nuts	No. of nuts	Length of one leg

SETS OF ROLLERS.

Rollers	No.	Diameter	Length between shoulders	Extreme length
Cross-rods	No.	Diameter	Length between shoulders	Extreme length
Side-bars	No.	Thickness	Width	Extreme length

FILLERS FOR PINS.

No.	External diameter	Internal diameter	Length

TURN BUCKLES AND SLEEVE NUTS.

No.	Taps

BRACING FRAMES FOR STRINGERS AND GIRDERS.

Angles	No.	Legs	Weight per foot	Extreme length
Plates	No.	Width	Thickness	Extreme length

DIAGONAL BRACING ANGLES FOR PLATE GIRDER SPANS.

No.	Legs	Weight per foot	Extreme length

WASHERS.

No.	Diameter	Diameter of bolts.

SEPARATE RIVETS.

No.	Diameter	Length under head	Kind of head	Position in bridge	Parts connected

PIN PILOTS.

No.	External diameter	Internal diameter

LOCK NUTS.

No.	Size	Description

Some engineers send also a complete bill of rivets to be used in the shop; but this is scarcely necessary, as it is more properly the place of the manufacturer to prepare such a bill.

The following form will be needed for the purpose:—

RIVETS.

Member	No.	Diameter	Length between heads	Kind of heads	Parts connected

An allowance of three per cent should be made for waste in shop rivets, and from ten to twelve per cent in field rivets.

If the hip verticals be flat bars, they are to be transferred to the group of "Main Diagonals, etc."

The corresponding form of "Shipping Bill" is as follows;—

STRUTS.

Member	No.	Length centre to end, or extreme length	Mark

BARS.

Member	No.	Section	Diameter of eyes	Sizes of heads	Length centre to centre of eyes	Mark

RODS.

Member	No.	Diameter	Diameter of eyes	Diameter of upset ends.	Threads R. or L.	Length centre of eye to end, or centre of eye to centre of eye	Mark

SIDE BRACING.

No.	Section	Extreme length	Mark

FLOOR—BEAMS.

No.	Extreme length	Mark

TRACK STRINGERS.

No.	Extreme Length	Mark.

ROLLER AND BED PLATES.

No.	Position (fixed or free end)	Mark, if any

NAME PLATES.

No.	Date

OTHER SEPARATE PLATES.

No.	Position	Mark

PINS AND THEIR NUTS.

No.	Diameter	Length between shoulders	Extreme length	Dimensions of ends	Mark

BOLTS AND THEIR NUTS.

No.	Diameter	Diameter of upset ends	Length under head, or extreme length

BRACKETS.

No.	Position	Extreme length	Mark

ORNAMENTAL WORK.

No. of pieces	Description	Mark

GUARD RAILS.

No.	Length.

BEAM HANGERS AND THEIR NUTS.

No.	Diameter of eye	No. of nuts and lock-nuts	Mark

ROLLERS.

No. of sets

FILLERS FOR PINS.

No.	External diameter	Internal diameter	Length	Mark

TURN BUCKLES AND SLEEVE NUTS.

No.	Taps

BRACING FRAMES FOR STRINGERS AND GIRDERS.

No.	Position	Mark

DIAGONAL BRACING ANGLES FOR PLATE GIRDER SPANS

No.	Position	Mark

WASHERS.

No.	Diameter	Diameter of bolt

SEPARATE RIVETS.

No.	Diameter	Length under head	Kind of head	Position in bridge	Parts connected

PIN PILOTS.

	No.	External diameter	Internal	diameter	Mark

LOCK NUTS.

No.	Size	Description	Mark

The following is the system of marking iron before shipment which the author would recommend. It should be thoroughly comprehended by the manufacturer, the foreman in charge of erection, and the time-keeper or clerk, if there be either employed on the work.

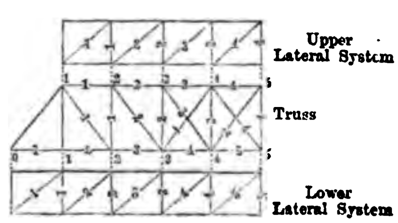
Where the work is very extensive, the time-keeper generally checks the material as it arrives on the ground.

First, if there be more than one span, each piece of each span should be marked with a daub of color peculiar to that span: thus the first span may be white, the second yellow, the third blue, etc.; care being taken to choose such colors as will be readily distinguished upon the iron-work.

The colors may be marked in the last column of each division in the "Shipping Bill" and at the end: thus the mark for a main diagonal may be "3 Bl.," or "2 W.;" the first representing the third set of main diagonals in the third span, and the other the second set in the first span. The letters Bl. are chosen for blue, so as not to be mistaken for the letter B used elsewhere. A similar precaution should be taken with the other colors.

In addition to this characteristic color-mark, each piece should be marked in white paint according to the following method.

R. and L. denote that the piece, if a main portion, lies to the right hand or to the left hand when one stands at the nearest portal, astride the centre plane of the bridge, and looks towards the centre of the span. If a detail, it denotes that it lies to the right or left hand when one stands astride the middle vertical plane of the truss to which the detail belongs, at the foot of the nearest batter brace, and facing the centre of the span. The numbers can be readily understood by referring to the accompanying diagram.



Chord sections are to be numbered, and marked R. or L.

Batter braces are to be marked R. or L.

Channel bottom chords are to be numbered.

Posts to be numbered, and marked R. or L.

Upper lateral struts to be marked U. 1, U. 2, &c.

Lower lateral struts to be marked L. 0, L. 1, L. 2, &c.

Portal struts to be marked U or L (upper or lower).

Intermediate struts to be marked In. 1, In. 2, &c., the numbers corresponding with those of the posts to which they are to be attached.

Main diagonals and counters to be numbered.

Hip verticals need no mark.

Upper lateral rods to be marked U. 1, U. 2, etc.; the numbers corresponding to those on the diagram.

Lower lateral rods to be marked L. 1, L. 2, etc.; the numbers correspondiug to those on the diagram.

Portal vibration rods to be marked P.

Intermediate vibration rods to be marked V.

Chord bars to be marked 1 A, 1 B, 1 C, 2 A, 2 B, 2 C, etc.; the numbers corresponding to those on the diagram, and the letters denoting the position in the panel, A being for those on the exterior side of the truss, B for those next to the outside, &c.

Side braces to be numbered to correspond to the panel points to which they belong, and to be marked R. or L.

Track stringers to be numbered to correspond with the panel, and to be marked R. or L., if there be any difference.

Floor beams to be numbered to correspond to the panel points.

Roller and bed plates to be marked R. or L., if there be any difference.

Name plates require no marks.

Separate plates to be numbered so as to correspond to the panel points to which they belong, and to be marked R. or L., if necessary.

Lower chord pins to be marked L. 0, L. 1, L. 2, etc.; the numbers corresponding to those of the panel points.

Upper chord pins to be marked U. 1, U. 2, U. 3, etc.; the numbers corresponding to those of the panel points.

Portal diagonal pins to be marked P.

Vibration-rod pins to be marked V.

Pins at middle of posts to be marked M. 1, M. 2, M. 3, etc.; the numbers corresponding to those of the posts.

Upper lateral rod pins to be marked T. 1, T- 2, etc., the numbers corresponding with the panel points.

Lower lateral rod pins to be marked B. 0, B. 1, B. 2, etc., the numbers corresponding with the panel points.

Bolts need no mark, but should be boxed before shipment.

Brackets to be marked P. or I. (portal or intermediate), also R. or L.

Ornamental work to be marked R. or L.

Guard rails require no marks.

Beam hangers to be numbered so as to correspond to the panel points to which they belong.

Rollers need no marks.

Fillers to be marked the same as the pins to which they belong.

Turn buckles and sleeve nuts, being attached to the rods before shipment, require no marks.

Bracing frames for stringers and girders to be numbered so as to correspond with the panel points and the upper sides to be marked U p.

The frames may be considered as dividing plate girder spans into panels.

Diagonal bracing angles for plate girder spans to be marked U or L (upper or lower) and to be numbered to correspond with the panels into which they may be supposed to divide the span.

Washers need no marks: they should be boxed, or strung on bolts, before shipment.

Rivets need no marks, but should be boxed.

Pilot nuts need no marks, as there are so few of them required.

Lock nuts should be marked D_1, D_2, &c., U_1, U_2, &c., $V._1$, $V._2$, &c., L. 1, L. 2 &c., H. 1, H. 2, &c. and P, D corresponding to web diagonal, U to upper lateral rod, V. to vibration rod, L. to lower lateral rod, H. to beam hanger and P to portal rod.

In addition to these marks, there should be others for those members which are to be riveted together in the field, and which are assembled in the shop when the rivet holes previously punched are reamed. These marks should be punched into the iron with a steel point, and should consist of one, two, three, or four dots upon, each of the pieces so assembled, in order that no piece during erection will be put into the wrong place.

CHAPTER XXII.

ERECTION AND MAINTENANCE.

The number of men required to erect an iron railroad bridge will vary from a dozen to a hundred and fifty or even more according to the length of span, location and the time to be occupied in erection.

For any one bridge, there is a certain number of men which will be more economical than any other number; and it is only experience which will enable one to tell beforehand what this number is.

If there are too few hands, the work will lag, and difficulty will be experienced in handling heavy pieces: on the other hand, if there are too many men, they will stand in each other's way, and the total amount of effective work done by each man per day will not be a maximum. If, for any reason, there be need of haste, it will be economical to have a large force of men, notwithstanding the last mentioned consideration.

Owing to certain well known peculiarties of Japanese workmen it is very difficult to say how many men will be needed in any particular case: this matter will have to be left almost entirely to the judgement of the engineer. To such as have had no experience in bridge erection, the author offers with much diffidence the following tables as a mere guide.

FOR RAISING SINGLE TRACK BRIDGES

SPAN	No. OF MEN REQUIRED
Plate Girders	From 12 to 18
Pony Trusses	„ 20 „ 25
Thro. Spans under 100′	„ 25 „ 30
100′ to 125′	„ 30 „ 35
125′ „ 150′	„ 35 „ 40
150′ „ 175′	„ 40 „ 45
175′ „ 200′	„ 45 „ 50
200′ „ 225′	„ 50 „ 60
225′ „ 250′	„ 60 „ 75
250′ „ 275′	„ 75 „ 90
275′ „ 300′	„ 90 „ 110

FOR RAISING DOUBLE TRACK BRIDGES

SPAN	No. OF MEN REQUIRED
Plate Girders	From 15 to 20
Pony Trusses	„ 25 „ 30
Thro. Spans under 100′	„ 30 „ 40
100′ to 125′	„ 40 „ 45
125′ „ 150′	„ 45 „ 55
150′ „ 175′	„ 55 „ 60
175′ „ 200′	„ 60 „ 70
200′ „ 225′	„ 70 „ 80
225′ „ 250′	„ 80 „ 95
250′ „ 275′	„ 95 „ 115
275′ „ 300′	„ 115 „ 135

If the bridge is to be erected rapidly the number given in the table can be probably advantageously increased by from twenty-five to fifty per cent.

The cost of raising a bridge depends more upon the foreman than upon the men. The best men will fail to do their full quota of work if the foreman be not energetic. Nor does it suffice to have simply a good worker for a foreman: he must know how to keep the gang busy, or they will stand by and look on, while he does the work. He should also have their good will, or the progress of the work will be unsatisfactory.

The outfit for a gang can be taken from the following table, in which the smaller numbers are for short spans, and the large numbers for long spans.

IMPLEMENTS.	NUMBER REQUIRED.
Forges	1 or 2
Pairs of tongs	2 to 5
Button setts for each size of rivets	2 to 5
Drift pins of each necessary size	5 to 15
Reamers	4 to 10
Handle cold chisels	2 to 5
Handle drift pins	2 to 4
Cape chisels	15 to 30
Plain chisels	8 to 16
Rachet drills	1 to 3
Wrenches for $\frac{5}{8}''$ nuts	5 to 10
Wrenches for $\frac{3}{4}''$ nuts	5 to 10
Wrenches for $\frac{7}{8}''$ nuts	5 to 10
Wrenches for 1″ nuts	3 to 6
Rivetting hammers	2 to 5
Light sledges	1 to 3
Heavy sledges	1 to 3
Hand lines $\frac{3}{4}''$ dia.	5 to 12
Guy lines 1″ dia. by 150′ long	5 to 12
Fall lines 1″ dia. by 150′ long	2 to 5
Rope slings	10 to 20
Sets of 10″ blocks	2 or 3
Sets of 8″ blocks	2 to 5
Snatch blocks	2 to 5
Steel crow-bars	8 to 20
Cross-cut saws	3 to 7
Augers 1″ dia.	3 to 6
Augers $\frac{7}{8}''$ dia.	3 to 6
Augers $\frac{3}{4}''$ dia.	3 to 6
Augers $\frac{5}{8}''$ dia.	2 to 4
Axes	5 to 15
Adzes	3 to 7
Timber trucks	10 to 25
Monkey wrenches	5 to 12
Chains	5 to 12
Crabs	2 to 5
Holding on bars	2 to 4
Jack screws	3 to 8
Large wrenches of different sizes for pins	2 to 6

and if necessary a pile driver with its appurtenances. The ordinary weight of a pile driver hammer varies from sixteen hundred to two thousand pounds, and will probably cost from two hundred to two hundred and fifty or even three hundred yen.

The height of the driver should be about thirty feet.

Plate XII. illustrates nearly all the tools found in the list. The author wishes to apologize for the appearance of this plate and its very apparent want of scale ; for it was prepared from clippings taken from various advertising circulars. Some of the figures do not represent exactly what they were originally intended for, but the agreement is exact enough to give the reader a clear idea of the form of each tool.

Besides the tools on the list, each carpenter should be provided with the usual special tools to be found in a carpenter's kit.

In getting ready to erect a bridge the first step is to prepare the ground in the neighbourhood of the site, so that there will be room to store the material and for the men to work. When the iron is received at the site it should be checked; and any pieces from which the marks have been obliterated should be remarked. The iron should be piled systematically, similar parts being grouped, and no iron should be allowed to lie upon the ground. It should be piled so that there will be no trouble in getting at any piece which may be required. The portions to be used first should be placed nearest the bridge site.

The piers and abutments will be supposed to be erected, as this work does not aim to treat of foundations.

The next step is to put the falsework in place. If the bed of the stream be dry or nearly so, the bottom hard, the distance from the bed to the lower chord be no greater than twenty feet, and if there be no danger of a sudden rise of water with a swift current, vertical timbers resting on foot blocks with a cap and light diagonal bracing may be employed.

If the ground be not perfectly firm, mud sills must be used instead of foot blocks, and, if at all soft, piles must be employed. The size of a mud sill should vary from 6″ × 6″ to 12″ × 12″ according to the hardness of the ground, the weight upon the sill and the height of the falsework. It is not necessary that the timbers be square: for ground not especially hard wide timbers laid on their flats are preferable, as they distribute the pressure better. Square timbers should be used where the ground is hard in some places and soft in others, so as to prevent unequal settlement and a consequent distortion of the bent.

If there be but one tier per bent, two vertical posts will be sufficient for short spans of single track bridges, but in all other cases a third vertical post midway between the others will be required. The caps should be from 6″ × 8″ to 8″ × 10″ according to their unsupported length and the magnitude of the weights which will come upon them: they should project two feet beyond each truss. The upper ends of the posts should lie directly under the trusses, and the caps should be drift bolted thereto. The bent should be braced by diagonal flat timbers, say from 2″ × 6″ to 8″ × 8″, according to their length, running in opposite directions, one on each side of the bent, and bolted or spiked to the posts and cap.

If there be two tiers in a bent or one tier resting upon piles which project more than five feet above the surface of the ground, inclined posts having a batter of two inches to the foot should be placed outside of the vertical posts under the trusses.

Each tier should be braced with diagonal timbers, as before. The greater the danger of high wind, the more effectively should each bent be braced. Alternate consecutive bents should also be braced diagonally on their outer faces, and all consecutive bents should be connected by longitudinal horizontal planks well spiked to the caps. These planks will be useful, in fact often necessary, for the workmen in passing from bent to bent. If there be more than two tiers per bent, the batter of the inclined posts should be three inches to the foot. A good height for each tier is sixteen feet.

Where the water is deep and rapid, piles will be required to rest the bents upon. There should be from two to five piles per bent, according to the width of the latter; a pile being placed below each vertical and inclined post. These piles should be braced in the direction of the stream by flat timbers bolted thereto. Any bracing that may be given them transversely to the stream should be at such a distance above high-water level as to cause no obstruction to boats, trees, ice, or other floating objects.

If the bottom be bare rock, incapable of holding piles, the mud-sills must again be resorted to. They should be weighted so that they may be sunk into place, then drift-bolted to the rock. This can be done without the aid of a diver. Of course the sills must be firmly attached to the lower tier before being put down.

The tops of all piles should be cut off to an exact level, so that, when the bents are erected, the upper surfaces of the upper caps will lie in the same horizontal plane.

On these caps should be placed timber-beams stretching from one bent to the next, and lying immediately under the trusses. It is generally customary to place the bents under the panel points; but the author prefers to put them two feet to one side, so that the floor beams may be swung into place without taking down the falsework. This method may, and probably will, require an extra bent at one end of the span; so, if the bents be expensive, it is better to put one under each panel point, and remove the upper tiers before swinging the floor-beams.

For spans where the track stringers rest upon the floor beams, or for any spans when the bents of falsework are directly under the panel points, the level of the top of the longitudinal beams should be at least twelve inches below the feet of the posts, so as to permit of the use of camber blocks, like those shown on Plate XI. The angle which the contiguous faces make with the horizontal (less, of course, than the angle of friction of the wood) enables the under block to be easily knocked out when the span is to be swung.

But for the case of stringers abutting against floor beams, and falsework bents to one side of the panel points blocks must be placed between the cambre blocks and the timber beams, so that the track stringers may pass over the upper caps of the lower falsework with a couple of inches clearance.

The timbers for the caps and posts of the falsework are generally square, and the sizes for the latter are to be found from Table XXIV., after the stresses in

them have beeu ascertained as follows: —

Let

W_1 = weight per foot of the iron-work of the bridge,

W_2 = average weight per foot in height of one bent of falsework and the timbers whose weight it supports,

p = wind pressure per square foot,

A = area per lineal foot which the two trusses present to the wind (it is generally about five or six square feet),

A' = the average area subject to wind pressure per foot in height on one bent, and its share of longitudinal bracing,

l = panel length,

c_1, c_2, c_3, etc. = horizontal distance between centre lines of inclined posts measured along the caps,

d = depth of truss,

d_1, d_2, d_3, etc. = heights of the different tiers commencing at the top,

h = vertical distance between centre of chord and upper cap of bent,

and

θ = the angle which the inclined posts make with the vertical;

then

pAl = pressure on trusses at each panel point,

$pA'd_1$ = pressure on upper tier,

$pA'd_2$ = pressure on second tier from top,

$pA'd_3$ = pressure on third tier from top,

and the stresses F_1, F_2, F_3, etc., in the inclined posts of the first, second, and third tiers respectively, will be given by the equations.

$$F_1 = \frac{p \sec \theta}{c_2}\left[Al\left(\frac{d}{2} + h + d_1\right) + \frac{A'd_1^2}{2}\right] + \left(\frac{W_1 l}{2} + \frac{W_2 d_1}{4}\right)\sec\theta,$$

$$F_2 = \frac{p \sec \theta}{c_3}\left[Al\left(\frac{d}{2} + h + d_1 + d_2\right) + \frac{A'(d_1 + d_2)^2}{2}\right] + \left[\frac{W_1 l}{2} + \frac{W_2}{2}\left(d_1 + \frac{d_2}{2}\right)\right]\sec\theta,$$

$$F_3 = \frac{p \sec \theta}{c_4}\left[Al\left(\frac{d}{2} + h + d_1 + d_2 + d_3\right) + \frac{A'(d_1 + d_2 + d_3)^2}{2}\right] + \left[\frac{W_1}{2} + \frac{W_2}{2}\left(d_1 + d_2 + \frac{d_3}{2}\right)\right]\sec\theta,$$

F_4 = &c. + &c.

These formulae are obtained under the supposition that the inclined posts are not aided by the vertical ones, which supposition is necessary in order to avoid ambiguity: it would be correct, were the falsework on the verge of overturning. If the timber be green, the error thus made upon the side of safety is advantageous; but, if the timber be dry and of good quality, it is permissible to make a slight reduction in the size given by Table XXIV. In applying the table, find the size of square imber required for a stress F_1 and length $d_1 \sec \theta$, that for a stress F_2 and length $d_2 \sec \theta$, etc., then take the greatest of these sizes.

The vertical posts should be strong enough to withstand a working-stress given by the equation,

$$S = \frac{W_1 l}{2} + \frac{W_2}{2}\left(d_1 + d_2 + \&c. + d_{n-1} + \frac{d_n}{2} \right),$$

where n is the number of the tier considered, and S the stress in the corresponding vertical post.

One dimension of the vertical posts should be the same as the side of the square which is the section of the inclined posts; so that the diagonal braces may be flush with the entire faces of the bents, and be bolted to the verticals without the intervention of filling-pieces.

The more elevated the bridge, the more important does it become to properly proportion the falsework. The values of W_2 and A' will have to be assumed, or roughly calculated, before applying the equations. The other quanities are, or should be, known. The value of p may be taken from ten to fifteen pounds per square foot, unless the situation be more than ordinarily exposed, when it may be taken at twenty pounds. One can afford to risk the chance of a hurricane striking the bridge before it is swung.

The sections of the caps are generally made the same as those of the inclined posts. The caps should be dapped to receive both upper and lower ends of vertical and inclined posts. The vertical posts should be drift-bolted through the caps, the bolt being long enough to project five or six inches into each post; and the inclined posts should be held in place by wooden splice pieces, one on each side of the bent, projecting above and below the cap, and fastened at each end by a bolt passing through the two splice pieces and the post. This attachment may be used for the vertical posts instead of the drift bolts.

For additional security against slipping a third bolt may be put through the splice pieces and the cap, or cleats may be nailed to the caps above and below at the toe of each inclined post.

All bolt holes in the timber should be accurately located and bored before the falsework is erected. On this account the bents should be all built after one pattern so that the parts may be interchangeble. If the bents be of different heights, the variation may be effected in the lowest tiers. Bolts are always preferable to spikes for connecting timbers, especially when the falsework has to be taken down and re-erected for another span. Care should be taken to avoid any unnecessary injury to the timber, in order that it may not be sold at too great a loss after the work is finished.

There should be two plank walks on top of the lower falsework, exterior to the trusses and a runway midway between formed of several wooden joists set on edge, for the purpose of bringing out the material thereon upon timber-trucks.

The following table taken from Carnegie's "Pocket Companion" gives the safe uniformly distributed loads in pounds for these joists. The safe concentrated central loads can be found by dividing those given in the table by two.

WOODEN BEAMS.

Safe Load, Uniformly Distributed, for Rectangular White or Yellow Pine Beams one inch thick,

allowing 1200 lbs. per square inch fiber strain.

To obtain the safe load for any thickness, multiply the safe load given in table, by the thickness of beam.

To obtain the required thickness for any load, divide by the safe load for one inch, given in table.

Span in Feet.	DEPTH OF BEAM.										
	6″	7″	8″	9″	10″	11″	12″	13″	14″	15″	16″
Feet.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs
5	960	1310	1710	2160	2670	3230	3840	4510	5230	6000	6830
6	800	1090	1420	1800	2220	2690	3200	3760	4360	5000	5690
7	690	930	1220	1540	1900	2300	2740	3220	3730	4290	4880
8	600	820	1070	1350	1670	2020	2400	2820	3270	3750	4270
9	530	730	950	1200	1480	1790	2130	2500	2900	3330	3790
10	480	650	850	1080	1330	1610	1920	2250	2610	3000	3410
11	440	590	780	980	1210	1470	1750	2050	2380	2730	3100
12	400	540	710	900	1110	1340	1600	1880	2180	2500	2840
13	370	500	660	830	1030	1240	1480	1730	2010	2310	2630
14	340	470	610	770	950	1150	1370	1610	1870	2140	2440
15	320	440	570	720	890	1080	1280	1500	1740	2000	2280
16	300	410	530	680	830	1010	1200	1410	1630	1880	2130
17	280	380	500	640	780	950	1130	1330	1540	1760	2010
18	270	360	470	600	740	900	1070	1250	1450	1670	1900
19	250	340	450	570	700	850	1010	1190	1380	1580	1800
20	240	330	430	540	670	810	960	1130	1310	1500	1710
21	230	310	410	510	630	770	910	1070	1240	1430	1630
22	220	300	390	490	610	730	870	1020	1190	1360	1550
23	210	280	370	470	580	700	830	980	1140	1300	1480
24	200	270	360	450	560	670	800	940	1090	1250	1420
25	190	260	340	430	530	650	770	900	1050	1200	1370
26	180	250	330	420	510	620	740	870	1010	1150	1310
27	180	240	320	400	500	600	710	830	970	1110	1260
28	170	230	300	390	480	580	690	800	930	1070	1220
29	170	230	290	370	460	560	660	780	900	1030	1180

The posts for the upper falsework should rest on the caps of the lower falsework a few inches inside of the trusses: they should be attached by splice timbers and cleats. The height of the upper falsework should be such that the upper surface of the cap will be at least six inches below the under sides of the top chord sections, so as to permit of the use of cambre blocks between.

The author would recommend that the end bents of upper falsework be made three or four feet higher than the others, and the use of four posts there instead of two, one on the inside and one on the outside of each truss, in order to aid in raising and retaining in place the heavy batter braces. After the latter are put in position a horizontal timber may be firmly bolted to the bent at the level of the other bent caps for the temporary flooring to rest upon. Stout beams stretching from bent to bent will be

required to act as fulcra for the levers by which the chord sections are lifted and held in place while being connected.

The cap of the upper falsework should be deeper than it is broad, because it has to act as a beam, and may be subjected to considerable shock, when the chord sections are being put in place. The method of bracing shown on Plate XI. is specially advantageous in respect to this consideration.

The upper falsework should be braced longitudinally as well as transversely.

The sizes of the posts will vary from 6″ × 6″ to 8″ × 8″ according to their length and the weight which they have to support.

In both the upper and lower falsework the diagonal bracing in planes parallel to the longitudinal axis of the bridge should for economy's sake be placed between alternate pairs of bents; that is, every other space between bents will be braced: the end spaces should, however, be braced in any case.

Plate XI. gives an illustration of how the working drawings for falsework should be made. For economy of space the scale has been taken at one quarter of an inch to the foot; but it should, if intended for an actual case of framing, be twice as great. A drawing of this kind should be accompanied by bills of lumber and iron prepared in a similar manner to that explained in Chapter XVII. Measurements of distances between bolt holes should be both scaled and calculated: those on Plate XI. were simply scaled, as the plate is intended for illustration only.

The foreman of the work should be furnished with a blue print of the working drawings for the bridge, unless the type of structure be one with which he is perfectly familiar. He must also be provided with a raising bill, which should consist of a skeleton diagram of one truss with the following information written thereon.

Size of each truss strut and tie and mark for same; also number of pieces of same in a panel of one truss.

Diameters and lengths *s.* to *s.* of pins with their marks.

Diameters, lengths and marks of fillers for same.

Sizes and marks of all separate plates belonging to the trusses, each in its proper position.

A diagram for the lower lateral system giving the following information;

Sizes and marks of rods
Positions of same showing which eyes are to go next the trusses.
Sections, lengths and marks of lateral struts.
Diameters and lengths of lateral pins, if any.
Diameters and lengths of fillers for same.

A diagram for the upper lateral system and portal bracing giving the following information;

Sizes and marks of rods.
Positions of same showing which eyes are to go next the trusses.
Sections and marks of lateral and portal struts.
Diameter and length of portal pins.
Diameters and length of fillers for same.
Diameter and length under head of portal strut attaching bolts.

He should be also provided with a plan of the bottom chord packing, the transverse dimensions being exaggerated so that the size of each piece may be written thereon; a bill of bolts giving the number and position of each kind, and a clear statement of the system of marking the iron.

Before starting to erect the bridge the foreman should study carefully all the plans so that he will have a clear picture of the bridge in his mind's eye, and will not have to be continually referring to the drawings during the erection. On a work of any magnitude there should be kept on hand a few standard nuts of each size ordinarily used, so that the loss of a nut or two will cause no delay: for the same reason there should be a few extra bolts of each size.

The material as a general rule is all piled on one side of the stream, the raising should therefore be commenced at the other side so that the passage of the material will not interfere with the work. If there be no objection, the far end of the bridge should be the fixed one, so as to start from something permanent, but this is not absolutely necessary.

To illustrate the method of raising take for example the bridge treated in Chapter XVIII., and assume that the foundations with their anchor bolts and falsework, are in place. The first thing to be done is to lay out the centre line of the bridge upon the falsework caps, marking it with a small-headed tack on each cap, then the centre lines for the trusses in the same way. This can be done either with a transit, or with a carpenter's chalk-line; care being taken to make the transverse measurements to the outer lines exactly perpendicular to the central line. A test of the accuracy of the perpendiculars can be made by the three, four, and five method, using a tape-line. Next, mark the exact positions of the panel points upon the longitudinal beams under the trusses, and place the camber blocks, levelling over them so as to make the lines joining the central points of their upper surfaces parallel to the curve of the chords. It is better to have the blocks a trifle high, say, a quarter of an inch near the centre, and an eighth of an inch near the ends, or more, if on account of the height of the falsework, a greater settlement be anticipated. It is better to have the height too great than too little; for, if on account of too much cambre the last chord connection cannot be made, it is only a few minutes' work to tap the blocks a little, so as to lower them the requisite amount.

Four small nails will hold each pair of camber blocks from slipping during the work, and they can be left so as to be easily extracted before swinging the bridge. Next transfer the centre lines of the trusses to the tops of the camber blocks, and mark accurately the first panel points from the fixed end, then, starting there, pack the chord bars of both chords. It might be convenient to have a few hard-wood pins to fit the holes pretty tightly, so as to aid in getting the bars properly placed longitudinally.

After the chord packing has made some progress, run out the two batter braces, and hoist them into place by means of pulleys attached to the cap of the first bent of falsework, which bent should have been previously guyed and braced so that it cannot possibly be disturbed by the effect of the pulleys. As soon as each batter

brace is raised, and the anchor bolts pass through the holes in the shoe plate; the nuts should be tightly screwed down in order to aid in holding the batter brace in position.

It will not do, however, to rely solely on these, for the threads of the end bolts might be stripped: consequently a hard-wood supporting block must be strongly bolted to the two adjoining posts of the bent of the upper falsework. This block should have a bevelled edge, the angle of bevel being equal to the slope of the batter brace, so that the iron-work will not rest on a sharp edge of wood. If the lattice bars interfere with the bearing, as they are liable to do, rough notches can be cut in a minute on the bevelled face so as to bring the bearing upon the channels.

Meanwhile the end lower lateral strut, the end lengths of the lower chord struts, the portal struts and the portal and end lower lateral rods, having been run out, they are to be put into place, the portal struts being retained there by their connecting bolts, and the lateral strut by the end chord pins, which should also pass through the chord bars, chord struts and the fillers.

As the portal rods are adjusted by turn buckles with single tap ends, they may be omitted until after the portal struts are in place.

Next run out, and hoist upon the falsework, by means of pulleys attached thereto and timbers used as levers, the end sections of the top chords, working them into place by the levers, and attaching them by the hip pins, which should also pass through the end main diagonals, hip verticals and fillers. The other ends of the chord sections rest on the cambre blocks.

Next run out, and hoist into place, the first vertical posts, letting the upper ends lie in the open ends of the chord sections, and the lower ends pass around the lower chord struts.

Next bring out the second sections of the top chords and the second set of diagonals. Raise the chord sections into place, as before, with pulleys and beam levers, holding them there until temporary bolts are put into a few holes through the connecting-plates, filling plates and channel webs, and until the pins are run through the posts, diagonals, and fillers.

Such small portions of the structure as pins, fillers, and beam hangers, should not be brought out upon the falsework until required for use, for fear of their being lost overboard. Nothing more will be said about running out these and other small portions, but it will be assumed that they will be at hand when wanted. It should be an understood thing between the foreman and the men, that any one who drops any portion of the bridge into the water forfeits a certain amount of his wages. Such an arrangement will make green hands a little more careful than they are apt to be generally.

Now start the rivet gang at work at the portal and let them follow up the work as it progresses.

Next run out and put into place, as before, the second pair of posts; then bring on the third sections of the chords, the third set of main diagonals, the first set of counters and the next lengths of the bottom chord struts, putting all into place as before, and so on until the end of the bridge is reached.

Just before the riveters complete the riveting of the portal, the first of the upper lateral and intermediate struts should be run out, and put into place; but the upper lateral and vibration rods should be omitted, as they would be in the way of the riveters, and can be readily inserted afterwards.

As the chord bars will be in the way of the rivetters when connecting the lower chord struts, the latter can be temporarily removed by withdrawing the pin from those on one side of the strut at the panel point where the connection is to be made and hoisting the bars out of the way until the connecting plate on that side is attached, then putting them into place again, and repeating the same operation on the other side of the strut. At these panel points it might be well to use wooden pins until the strut connection is made.

About the time that one-half the span is erected, commence running out the lower lateral struts and rods, putting them into place, inserting the hip verticals and fillers, and coupling the lower chords into their final position, leaving the beam hangers lying horizontally, so that, when the longitudinal supporting-timbers are removed, they will drop into their proper places.

A little before the riveters reach the end of the span, the upper lateral and vibration rods should be put into place, and screwed up *about* the right amount.

When the end of the bridge is reached by the riveters, the last couplings of the bottom chords can be made at the pedestals.

The shoes rest upon the rollers, which should have been put in *exactly* transverse to the direction of the bridge, and blocked so that they cannot move.

The last connection for each truss can be easily made by raising the hip either with levers or by jack-screws, and either pressing against the shoe with jack-screws abutting against blocks chained to the roller plate, or by attaching a pair of blocks to the pedestal and first panel point lower chord pin.

After the final coupling has been made, and the riveting is finished, knock out the upper chord camber blocks, so as to bring all the weight of the upper part of the bridge upon the posts; then take down the upper falsework.

Next knock out the camber blocks of the lower chords, lowering them together gradually so as to bring no shock upon the bridge, and remove the beams that supported the trusses.

The arrangement of the cambre blocks will generally have left sufficient headway between the lower lateral struts and the runway to allow the floor beams and track stringers upon timber trucks to pass between; but if not it will be very easy to construct a new runway by blocking up the middle of each lower lateral strut from the old runway and laying a line of planks from strut to strut: should the planks spring too much, they can be blocked up at their middle points.

Next run out the end track stringers with their bed plates and bracing frames and leave them on the pier, then bring on the end floor beam, and take up as much of the runway as is necessary to get it into place. It should be lowered beneath the ends of the hangers then raised into place, after which the filling plates should be placed on top, the hanger plates below and the nuts screwed up tightly and locked. Now get the end stringers into place, removing the runway and inserting the bracing

frames. The stringers can rest upon the supporting brackets until it is convenient to rivet them to the beam. Next run out the second pair of stringers putting them in place at the far end and supporting them from the lower strut temporarily. Then bring out the next floor beam, swing it into place removing the runway as it interferes with the work, and so on until the end of the bridge is reached.

As soon as the second floor beam is in place the rivetters can commence connecting the stringers to the beams, and follow up the work as fast as these members are laid in place.

When the track stringers rest upon the floor beams, quite another method of erection must be pursued. The stringers must be brought out at the same time as are the lower lateral struts and rods, both of which pass through holes in their webs. These holes must be large enough to give considerable play, both to allow for deflection under passing loads, and to facilitate the passage of the lateral system. The stringers should be supported temporarily upon blocks from the caps of the lower falsework, and can be made continuous from end to end of span by rivetting on the splice plates and bracing frames, removing of course, the runway.

After swinging the span, the stringers must still rest upon the blocks until the floor beams are put in place and screwed up: the floor beams can be run out upon the top of the stringers. Then the stringers and beams must be rivetted together by their connecting plates and brackets.

Next screw up every adjustable rod to the proper tension, which can be ascertained by the sound they make when tapped with a hammer.

Next wash off any mud or other impurity that there may be on the iron-work, and give it two good coats of paint wherever the brush will reach. The best kinds of paint to use are lead paints, when they can be obtained unadulterated; but they are at the same time the most expensive of all the paints used for iron-work. Iron oxide is a good paint, but requires more frequent renewal. The color should be such as to readily show any sign of rust: various shades of gray are efficient in this respect, and are at the same time pleasing to the eye.

Next put on the shims, ties, rails, guard rails and foot planks with their connections; a matter so simple as to require no explanation.

In long bridges of several spans, it may be economical to dispense with the upper falsework by using a travelling derrick, running upon wooden stringers, for the purpose of handling the heavy sections. Under these circumstances, the whole of the portal might be connected while lying upon the falsework, then hoisted into place in one piece, and supported there by shore timbers from the first bent of falsework. The bridge should be completed as the traveller retreats: otherwise there will be difficulty in carrying the members past the traveller. The material should be brought on cars within reach of the derrick.

The last thing to be done is to take down the falsework, and draw the piles from the bed of the stream. The latter is easily accomplished by a crab on the bridge; the rope being attached to the head of the pile, which is vibrated transversely in all directions while being lifted by the tension of the rope.

The popular idea that an iron bridge when once erected will last forever without

care is a fallacy. The unlimited duration of iron subjected to the shock of heavy and rapidly passing loads is not yet proven, and experience has shown that the ironwork of bridges requires care and attention as thorough as does that of locomotives.

Iron bridges should be thoroughly inspected for rust spots at least once a year; and, if any be found, the bridge should be repainted. One or two spots in places where something might have rubbed off the paint may be touched up with a brush; but, generally speaking, when rust spots begin to appear, it shows that two good coats of paint are required immediately.

The adjustable members should be tested occasionally by tapping with a hammer. This duty should not be intrusted to an ignorant workman, who may turn away on the nuts until he breaks the rods. Whenever, in passing over a bridge, any of the iron-work rattles, it shows that something is out of adjustment. Generally speaking, a well-proportioned iron bridge will not get out of adjustment unless some one meddles with the nuts or turn buckles.

The following extract from a paper on "The Preservation of Iron Bridges" by Mr. E. Paschen, a translation of which is given in the "Abstracts of the Institution of Civil Engineers" will give some useful hints as to how bridge inspection should be made.

"The Society of Architects at Berlin has directed its attention to the question, and proposes that the railway companies generally should institute a system of periodical inspections and reports as to the condition of their various iron bridges, and recommends that the observations should be divided into two classes, the first (general) to be made in respect of every bridge, and the second in special instances only.

'The general observations to be made every five years to include—

'1. Measurement of permanent deflection.

'2. Measurement of deflection caused by loading (at rest).

'3. Enumeration of those portions of the structure and rivets which may have already been renewed.

'4. Careful examination of plates at junctions of bracing with booms.

'5. Careful examination of paint and those places affected by rust.

'The special observations (to be made annually) to include—

'6. Deflection of the lower flange under moving load.

'7. Distance apart of top and bottom flanges.

'8. Length of the diagonals.

'9. Lateral distortion and vibration at the centre of the girders.

'All observations upon the stsucture when repeated, to be, if possible, made by the same inspector.'

In modification of the above the author suggests that the result of observations made by mere inspection should be kept separate from those obtained by loading, as the former could be made at any time at a comparatively slight expense, and the most important of the defects discovered, whereas the latter would necessitate the presence of a sufficient number of engines of the heaviest class, and for the time be-

ing stop all traffic; he therefore proposes that subordinates should be first carefully instructed under the superintendence of the chief inspector, and that afterwards it should be their duty frequently to examine the structures, a formal report from personal observation being made him once in two years, and that the load-test should be employed only once in ten instead of five years.

The special observations, it is suggested, should include the effect of temperature upon length of the girders, the amount of movement in the roller bed-plate* with trains moving in both directions, the comparative distances apart of the web verticals, measured near the top and bottom flanges when the girder is loaded, and the lateral deflection caused by wind pressure under the conditions of a loaded and unloaded girder.

The author recommends that a book should be kept for the entry of the inspectors' reports, the information being under the following headings viz: name, short description, and, where possible, the calculation of the strains,† and a general sketch with details of the most important parts; weights of iron in the construction, total weight of superstructure; details as regards the history of the construction, name of maker &c., character of the materials, and results of experiments as to strength, amount of deflection under moving and fixed load &c."

---

* According to the nomenclature of this treatise this word should be shoe-plate.

† Stresses.

CHAPTER XXIII.

EFFECT OF BRAKES ON BOTTOM CHORDS.

After reading this treatise thus far some Japanese engineer may still have an idea that some portions of the bridges designed are too light, more especially the bottom chords. This is due to preconceived notions caused by the stiff lower chords of the Japanese bridges. The chords here designed were proportioned for the combined effects of the maximum live load and the wind pressure: they can only be too light therefore, in resisting the shock of passing loads. Let us investigate this point. The most destructive effect of a train would be when it is allowed to come upon the bridge with all the brakes set. To be on the side of safety let us assume a train of engines, and take the coefficient of friction between wheels and rails to be 0.8. Michael Reynolds C.E. in his treatise on "Continuons Railways Brakes" p. 207 makes the actual maximum value of this coefficient 0.25, although two pages farther on he assumes it as we have done to be 0.8. With the train covering the whole span the combined live and dead load tensions in the lower chords would be so great that the thrust of the braked wheels would most assuredly be insufficient to overcome it, while, when the first pair of wheels comes upon the bridge, the thrust which they produce will also be too small to counteract the dead load tension. Between these two extremes there is one position of the loading which will be more effective than any other position in causing thrust upon the chords. The train should be brought on at the expanding end of the span, in which case the part upon which the thrust will act with greatest effect will be the end panels of the lower chord at the fixed end.

Let the dead load stress in the end panel of the bottom chord of *one truss* be denoted by T.

Let the variable reaction of the live load on *one truss* at the fixed end be represented by R,

then the corresponding end panel bottom chord stress will be $R \tan \theta$

where θ is the inclination of the batter brace to the vertical.

Let w = the uniformly distributed live load per lineal foot on *one truss*

f = the coefficient of friction between wheels and rails

l = length of span.

and x = length covered by the moving load, the origin of coordinates being taken at the expanding end of the span.

The reaction

$$R = \frac{wx^2}{2l},$$

and the corresponding end panel chord stress

$$= \frac{wx^2}{2l} \tan \theta$$

The thrust (neglecting the partially compensating rolling friction at the expanding end of the span) is $f w x$. The compressive stress on the chord at the fixed end, if there be any, will consequently be given by the equation

$$C = fwx - \frac{wx^2}{2l} \tan \theta - T$$

Differentiating to find a maximum gives

$$\frac{dC}{dx} = fw - \frac{wx}{l} \tan \theta = o \text{ and } x = \frac{fl}{\tan \theta}$$

Differentiating again gives

$$\frac{d^2C}{dx^2} = -\frac{w \tan \theta}{l}, \text{ in which } x$$

appears to the zero power, so that, when

$$x = \frac{fl}{\tan \theta}$$

is substituted therein, the second differential coefficient is negative, denoting a maximum.

Substituting

$$x = \frac{fl}{\tan \theta}$$

in the equation giving the value of C, gives for a maximum value of the compression

$$C_m = \frac{wf^2l}{2 \tan \theta} - T$$

The value of w deduced from the Chapter on "General Specifications" is 0.67 ton say 0.7 ton per lineal foot.

First let us try the 100′ span where T = 10.9 tons and $\tan \theta = 1$

$$\text{therefore } C_m = \frac{0.7 \times 0.8 \times 0.8 \times 100}{2} - 10.9 = -7.75 \text{ tons.}$$

showing that the thrust cannot overcome the tension.

Again taking the 140′ span, we have T = 16.97 and $\tan \theta = 0.87$, which gives

$$C_m = \frac{0.7 \times 0.8 \times 0.8 \times 140}{2 \times 0.87} - 16.97 = -11.9 \text{ tons.}$$

showing that the tendency to buckle the chord decreases as the span increases. Finally let us try a 70′ span through bridge, which is the one least fitted to resist the thrust. Here T = 6.66 tons, and $\tan \theta = 0.988$, hence

$$C_m = \frac{0.7 \times 0.8 \times 0.8 \times 70}{2 \times 0.988} - 6.66 = -4.3$$

showing that even in this case there is ample tension.

Hence we may conclude that the bridges which we have designed are fully capable of properly resisting any stress or combination of stresses to which they can ever be subjected.

CHAPTER XXIV.

RECAPITULATION.

Before closing this treatise it may be advisable to give a *résumé* of the various steps to be taken by an engineer in designing and building a bridge. They are as follows:—

1°. Ascertain as much as possible of the list of data in Chapter XVII., so as to know what kind of bridge is required, and what are the peculiarities of location that may affect its construction,

2°. Determine the live load, dead load, number of panels, depth of truss, engine excess, and wind pressure on each panel point of top and bottom chords when bridge is both empty and loaded.

3°. Fill out the table of data given in Chapter VIII.

4°. Find stresses in trusses by method of Chapter VIII., recording them on a skeleton diagram.

5°. Proportion main members of trusses recording dimensions on diagram.

6°. Determine from the tables sizes of members of lateral systems, floor systems, portal bracing and vertical sway bracing, and write them upon diagram.

7°. Proportion pins and write their diameters on diagram.

8°. Make out bills of materials, proportioning the details as they come in order upon the list of members.

9°. Check dead load.

10°. Make estimate of cost.

11°. Make working drawings.

12°. Make order and shipping bills and send to manufacturer together with explanation of methods of marking iron.

13°. Check all materials when received at site and pile them up in a convenient manner.

14°. Erect the bridge.

ADDENDA.

Since the preceding pages were written the author has seen in an otherwise very favourable review of his treatise on "The Designing of Ordinary Iron Highway Bridges" by the American Engineer a serious objection to the usual attachment of a floor beam by four hangers.

In the words of the review "the inner loop will take nearly, if not quite, all the load at the panel point; when the bridge is first adjusted; and this not only becomes constrained itself but also overstrains the inner tension brace.* The number of inner hangers which are constantly working loose, presumably by stretching, in railroad bridges in which this detail is used, demonstrates its unsatisfactory character."

The author has long recognized the inequality of distribution of floor beam load between the inner and outer hangers, but considered that the low intensity of working stress on these members would compensate for the objectionable inequality. Such has been also in all probability the opinion of most American engineers; for beams, when not rivetted to the posts, are nearly always suspended by four hangers. The fact of the inner hangers working loose can have been only lately discovered: it shows, however, that this detail needs improvement; and as the aim of this treatise is to design structures not only equal but in some respects superior to the best American bridges, it becomes necessary, even at this late hour, to correct the newly discovered fault.

The simple method of using single beam hangers will not always work, owing to the great bending moments which they produce upon the pins. For instance take the case of a double track bridge with panels twenty-four feet in length. The weight supported by each single hanger would be about forty tons, and the distance between centres of main diagonals would not be far from twelve inches. These data give a vertical bending moment upon the pin equal to one hundred and twenty inch tons, which *alone* would require an iron pin five and a half inches in diameter, or a steel one of four and three quarter inches; but when combined with the horizontal moment would call for a pin much larger than any intelligent designer would think for a moment of using.

The double hangers in such a case are a necessity, but the connection must be such as to distribute the load *equally* upon them. Such a distribution can be assured by using the following detail.

On the under side of the beam at each end is attached by four rivets a plate about five eighths of an inch thick, six inches long and as wide as or a little wider than the beam flange. This plate is placed symmetrically to the plane of the truss and the middle of the under side is grooved so as to receive one sixth of the surface of a pin about two inches in diameter, which rests in a similar groove on the top of the beam hanger plate. The lateral dimensions of this plate will be slightly greater than usual, but the thickness need not exceed one inch. To prevent the plate from

* Main diagonal.

rupture by bending there are attached to the underside by countersunk rivets two angle irons or plates bent into the form of angle irons, the vertical legs being connected by countersunk rivets, which in the neighbourhood of the pin pass as nearly as may be through the neutral surface of the T beam, and elsewhere near the lower edges of the angles.

As the axis of the pin is parallel to the length of the bridge, thev ertical legs must be transverse thereto.

This detail will be readily understood from the accompanying diagram.

To illustrate how to find the sizes of the stiffening plates, number of rivets required &c, it will be well to design a beam hanger plate for a 24′ panel of a single track bridge.

The total weight on the four hangers is about forty tons, and the centres of the beam hanger holes may be assumed to be situated on the corners of a six inch square. This would make the bending moment on the plate thirty inch tons, which would be resisted by the T shaped section of the two bent plates combined with the uncut portion of the beam hanger plate below the pin. Assuming the latter thickness ½″, and the plate stiffeners to be of 6″ × 6″ × ½″ angle iron would make the T about 12″ × 6½″ × 1″, the centre of gravity of which is about 5″ above the bottom.

The moment of inertia is therefore $\frac{1}{12} \times 12 + 12 \times (1.0)^2 + \frac{1}{12}(5.5)^3 + 5.5 \times (2.25)^2 = 54 +$

The resisting moment is given by the equation

$$M = \frac{RI}{d_1}$$

so taking $R = 4$ tons

$$M = \frac{4 \times 54}{5} = 43.2$$

As the bending moment was 30 inch tons, the sizes assumed are ample.

It will be well to use three quarter inch conntersunk rivets (the largest possible), so as to make the different portions of the T head act together.

There is a tendency to bend the plate in a direction at angles to the one considered, the moment for which is fifteen inch tons on each side of the pin. This will be resisted by a couple whose forces act as compression on the top plate of the T and tension on the rivets near the bottom of the angles. Taking the centre of moments at the middle of the top plate and the distance therefrom to the horizontal centre line of the rivet holes as $4\frac{3}{8}$ inches, will make the tension on the rivets $\frac{15}{4.38} = 3{\cdot}42$ tons. Using an intensity of only two and a half tons, because of the initial tension on the rivets, will make the section required 1.37 square inches; consequently two 1″ rivets will be sufficient.

This size of stiffened beam hanger plate may be adopted for all panels of single track bridges, or the thickness of the angles may be reduced to three eighths of an inch for short panels.

The difference in the total weight of iron per lineal foot caused by the use of

this detail will be from six to eight pounds for single track bridges, and from ten to fifteen pounds for double track bridges, the smaller numbers being for short spans and the larger for long ones.

It will be noticed in the diagram that the floor beam stiffeners at the support are placed close together so as to take up the vertical reaction of the hangers transferred by the auxiliary pin. The sectional area of these stiffeners should be about equal to that of the hangers.

Since this treatise was written, the author has prepared for the Institution of Civil Engineers, London, a paper entitled " An Analysis of the Weights of Iron and Dead Loads for Iron Pratt and Whipple Truss Railroad Bridges", the following deductions from which will be found useful to Japanese engineers.

For long single intersection deck trusses the economic depths are about one foot less than those for the corresponding through trusses: for short double intersection trusses they are about three feet less, and for long double intersection trusses about one foot less.

The ratios of the total weights of iron per lineal foot for single track deck and through bridges, *excluding the weight of the iron bents over the piers and abutments*, are given in the following table.

Span.	Ratio.	Span.	Ratio.
60'	0.88	180' S.I	1.01
80'	0.94	180' D.I	1.08
100'	1.00	190' S.I	1.01
150'	1.03	200' D.I	1.07
160'	1.04	250'	1.05
170'	1.02	300'	1.03

The ratio of total weights of iron per lineal foot for double and single track *through* bridges are as given in the following table.

Span.	Ratio.	Span.	Ratio.
60'	2.00	180' D.I	1.92
80'	2.00	200'	1.87
100'	1.94	240'	1.85
150'	1.91	280'	1.83
180' S.I	1.90	300'	1.82

The ratio of total weights of iron per lineal foot for double and single track *deck* bridges, *when the weights of the iron bents over the piers and abutments are not considered* are as given in the following table.

Span.	Ratio.	Span.	Ratio.
60'	1.99	180' D.I	1.74
80'	1.92	200'	1.74
100'	1.83	240'	1.74
140'	1.76	280'	1.74
180' S.I	1.76	300'	1.73

GLOSSARY OF TERMS.

Adjustable Member. — A member of a bridge the length of which can be increased or diminished at will.

Adze. — A tool for cutting timber. (Plate XII., Fig. 8.)

Anchor Plate or Anchor Piece. — A bent plate for holding down the expanding pedestal; Plate II, Fig. 12.

Angle Iron. — Iron rolled into the shape shown in section on Plate II., Fig. 3.

Apex. — The intersection of a brace with a chord or flange; called also a panel point.

Axis of Symmetry. — A line dividing an area into two parts equal and similar to each other, and similarly disposed to the line.

Bar. — A piece of iron flat or square in section.

Batter. — Slope, or inclination, to the vertical; usually measured by the tangent of the angle, or so many inches to the foot.

Batter Brace. — The inclined end post of a bridge. (Plate 1.)

Beam. — A member intended to resist bending.

Beam Hanger. — A rod or square bar supporting a floor beam from a chord pin. (Plate I. and Plate II., Fig. 10.)

Beam-hanger Nuts. — Nuts on the ends of beam hangers, serving to press the floor beam against the feet of the posts or against the chord heads. (Plate II., Fig. 10.)

Beam-hanger Plate. — A plate placed beneath the end of a floor beam for the beam-hanger nuts to rest against. (Plate II., Fig 10.)

Bearing. — A resting-place, usually for a pin or rivet.

Bearing-Pressure. — The pressure on a bearing.

Bed Plate. — A plate to distribute pressure upon masonry. (Plate IX, Fig. 14.)

Bending-Moment. — The moment of a force or forces which bend or tend to bend a piece.

Bending-Stress. — The stress produced in a piece by bending.

Bent. — A frame of timber or iron, usually the former, as a bent of false-work.

Bent Eye. - An eye on the end of a bar, the plane of which makes an angle with the direction of the length of the bar. (Plate II, Fig. 11.)

Bevel. — The slope on the end of a piece.

Bill of Material. — A list of various portions of material giving dimensions and weights, or other quantitative measurements.

Block. — A system of one or more pulleys or sheaves, so arranged in a frame or shell as to multiply the power of the rope passing around them, or to change its direction. (Plate XII. Fig. 2.)

Board Measure. — The measure of timber, the unit being a piece one foot square and one inch thick. Timber is sold at so much per thousand feet board measure, usually written, per M. b. m.

Bolt. — An iron rod with a square head at one end, and a thread and nut at the other.

Boom. — The English name for chord.

Brace. — Generally a strut, but sometimes the term is applied to a tie.

Bracket. — A knee or knee brace to connect a post or batter brace to an overhead strut. (Plate I. or Plate II., Fig. 11.)

Bracing Frame. — A frame to brace or stiffen parallel stringers or girders. (Plate II., Figs. 10 and 14.)

Built-Beam. — A beam made up of plates and angles riveted together. (Plate II., Fig. 10.)

Built Channel. — An assemblage of plate and angles in the form of a channel. (Plate

IX., Figs. 3 and 6.)

Burr. — A rough edge or ridge left by a tool in cutting metal. The term is sometimes used for a nut.

Button Sett. — A tool for forming the heads of rivets. (Plate XII., Fig. 5.)

Camber. — The upward curvature of a truss. It is measured by the height of the middle point of the centre line of the lower chord above the line joining the centres of end pins.

Camber Blocks. — Blocks of wood used in erection, so placed as to be easily removed. (Plate XI.)

Cape Chisel. — A tool for cutting iron. It consists of a rounded edge on the end of a short rod. The edge is very obtuse, so as not to break easily.

Centre of Gravity. — That point of a body about which the weights of all the different portions balance.

Channel, or Channel Bar. — Iron rolled into the shape shown in section on Plate II., Fig. 1.

Check Nut, or Lock Nut. — A contrivance to prevent a nut from turning when subjected to shock.

Chord. — The upper or lower part of a truss, usually horizontal, resisting compression or tension. (Plate I.)

Chord Bar. — A member of the chord which is subjected to tension. (Plate I.)

Chord Head. — The enlarged end of a chord bar, through which the pin passes.

Chord Packing. — The arrangement of the bottom chord of a truss.

Clear Headway. — The vertical distance from the upper surface of the rails to the lowest part of the overhead bracing. It is a measure of the height of the highest car that could pass through the bridge.

Clear Roadway. — The horizontal distance, measured perpendicularly to the planes of the trusses, between the inner edges of the batter braces. It is a measure of the width of the widest car that could pass through the bridge.

Cleat. — A narrow strip of wood nailed to something for the purpose of keeping a piece of work in its proper place.

Co-efficient of Friction. — A numerical quantity, which, multiplied into the normal pressure, gives the frictional resistance. It is equal to the natural tangent of the angle of repose.

Cold Chisel. — A tool for cutting iron. (Plate XII., Fig. 12.)

Column. — A pillar or strut; a long member which resists compression.

Component. — One of the parts into which a stress may be resolved or divided.

Compression. — A stress which tends to shorten the member which is subjected to it.

Concentrated Load. — A load which is, or may be considered, collected at one or more points.

Connecting-Plate. — A plate used for connecting two pieces.

Continuous Spans. — Consecutive spans connected over the points of support.

Counter. — An adjustable diagonal which is not subjected to stress by a uniformly distributed load covering the bridge. (Plate I.)

Countersunk Rivets. — Rivets, the heads of which are let into one or both of the plates which they connect, so as to leave a flush surface or surfaces.

Couple. — Two equal and parallel forces not acting in the same line.

Cover Plate. — A plate used to cover a joint, or to connect two pieces of the top chord plate. (Plate II., Fig. 11.)

Crab. — A slow-motion machine, worked by a crank for the purpose of winding a rope upon a drum, thereby raising a heavy weight. (Plate XII., Fig. 1.)

Crow Bar. — An iron lever. (Plate XII., Fig. 17.)

Curvature Stresses. — Stresses produced by the centrifugal force of passing trains, when the bridge is on a curve.

Dap. — To notch timber onto its bearing.

Dead Load. — The weight of all the parts of the bridge itself, and any thing that may

remain upon it for any length of time.

Deck Bridge. — A bridge in which the passing loads come upon the upper chords or the upper ends of the posts.

Deflection. — Motion laterally, or at right angles to the length of the piece. It is also used for the amount of motion, and is generally expressed in inches.

Depth of Truss. — The vertical distance between the centre lines of upper and lower chords.

Diagonal. — A member running obliquely across a panel. In this work all the diagonals except the batter braces are tension members.

Diagram of Stresses. — A skeleton drawing of a truss, upon which are written the stresses in the different members. (Plate XIII.)

Ditching Apparatus. — A contrivance for throwing a derailed vehicle clear of the track (Plate VI.)

Double Intersection. — The style of truss where the diagonals cross the posts at the middle of their length, as in the bridge shown on Plate I.

Double-riveted Lacing. — Lacing in which each bar is connected by two rivets at each end. (Plate II., Fig. 10.)

Double Tee. — Another name for I-beam.

Drift Bolt. — A round or square piece of iron, usually from one to three feet long, without head or nut, used to connect timbers.

Drift Pin. — A slightly tapering rod of hard steel, used for making rivet holes coincide. Its use is more convenient than advisable. (Plate XII., Fig. 14.)

Effective Area. — The gross area of a section, less that lost by rivet or pin holes; the net area.

Elastic Limit. — That intensity of stress at which the ratio of stress over strain commences to show a decided change. For wrought-iron it is from twelve to fifteen tons.

Erecting-Bill. — A bill of material for a bridge, so arranged as to facilitate the finding and placing of members during erection.

Expansion Joint. — The connection of pedestal to bed-plate, shown on. (Plate II. Fig. 15.)

Expansion Rollers. — A set of half a dozen or more turned rods of exactly the same diameter, placed under the shoe plate at one end of a truss to permit of expansion and contraction. (Plate II., Fig. 16.)

Extension Plate. — A plate riveted to the end of a strut channel, and projecting beyond it, to permit of the passage of a pin. (Plate II., Fig. 9.)

Eye. — A hole in the end of a member to permit of the passage of a pin.

Eye Bar. — A bar with an eye at each or one end.

Factor, or Factor of Safety. — The ratio of ultimate load to greatest allowable working-load. This term is getting out of favor among engineers, as its use has been somewhat abused. There is no such thing as a factor of safety for a well-proportioned bridge, for each member should have an intensity of working-stress proportionate to the character and amount of work which it has to perform.

Fall Line. — A rope used in erection for raising and lowering weights.

Falsework. — Temporary timber work to support a bridge during erection.

Field Riveting. — Riveting done in the field, or during erection. It is the poorest and most expensive kind of riveting.

Fixed End. — An end of a strut so firmly connected as to prevent all motion of the strut in the neighborhood of the end.

Filling-Plate. — A plate the function of which is to make flush two surfaces. (Plate II., Fig. 11.)

Filler. — A small ring of iron or piece of pipe placed on a pin in order to keep in position the members coupled thereon. (Plate II., Fig. 11.)

Fixed Load. — A load remaining permanently, or for a considerable length of time, upon a structure or portion of a structure.

Flange. — The upper or lower chord of a beam. It is the principal part for resisting either compression or tension.

Flexure. — Bending.

Floor System. — That part of the bridge which directly receives the travel.

Floor Beam. — A beam to support a portion of the floor and its load. (Plate I. and Plate II., Fig. 10.)

F. O. B. — Free on board ship, a term used in speaking of freight.

Foot Plank. — A plank for walking upon. (Plates I. and III.).

Forge. — An apparatus for heating iron. (Plate XII., Fig. II.)

Framing. — The carpenter work on timber.

Girder. — Any structure to cross a chasm or opening. The term is generally applied to short structures for places where it is not advisable to use trusses; for instance, a plate girder, or a rolled girder.

Guard Rail. — A rail to prevent a derailed train from running off a bridge. (Plate II., Fig. 14.)

Guys, or Guy Lines. — Lines for bracing the top of a pole, derrick, or any similar apparatus.

Gyration. — See radius of gyration.

Hammered Head. — A head formed on the end of a bar by hammering.

Hand Lines. — Small ropes used in erection.

Headway. — See clear headway.

Hinged End. — An end of a strut connected only by a pin.

Hip. — The place at which the top chord meets the batter brace.

Hip Joint. — The joint of the top chord and batter brace.

Hip Vertical. — A rod hung from the pin at the hip for the purpose of suspending the floor beam. (Plate I.)

Holding on Bar. — A lever to hold against one end of a rivet while the head at the other end is being formed with a button sett. (Plate XII., Fig. 16.)

I-Beam. — A piece of rolled iron of the section shown on (Plate II., Fig. 2.)

Initial Tension. — The tension caused in any adjustable member by screwing up the adjusting apparatus.

Intensity. — The intensity of a stress is the amount of stress upon a square inch of section.

Intermediate Strut. — An overhead strut in high bridges, attached to the posts of opposite trusses, and lying between the upper lateral strut and the floor. In deck bridges, if used at all, it would lie between the upper and lower lateral struts. (Plate I.)

Jack Screw. — A machine for raising heavy weights. (Plate XII., Fig. 10.)

Jaw. — A connection on the end of a strut similar to that shown on Plate II., Fig. 11., and Plate VIII., Figs. 7—10.

Joint. — A place where two abutting or lapping pieces are connected.

Joist. — A beam.

Knee or Knee Brace. — See bracket.

Lacing. — A system of bars, not intersecting each other at the middle, used to connect the two channels of a strut in order to make them act as one member. (Plate II., Fig. 11.)

Lacing-Bar. — A bar belonging to a system of lacing.

Lateral Rod. — A tension diagonal of a lateral system. (Plate I.)

Lateral Strut. — A compression member of a lateral system. (Plate I.)

Lateral System. — A system of tension and compression members forming the web of a horizontal truss connecting the opposite chords of a bridge. Its purposes are to transmit wind pressure to the piers or abutments, and to prevent undue vibration from passing loads.

Latticing. — A system of bars crossing each other at the middle of their lengths, used to connect the two channels of a strut in order to make them act as one member. (Plate II., Fig. 9.)

Lattice Bar. — A bar belonging to a system of latticing.

Leg. — One of the two portions of an angle iron separated from each other by the bend.

Lever Arm. — The perpendicular from the centre of moments to the line of action of a force. The lever arm of a couple is the perpendicular distance between the lines of action of the two equal and parallel forces.

Live Load. — The moving or passing load upon a structure.

Linville Truss (also called "Double Quadrangular," "Whipple." and "Double System Pratt" truss). — A truss with vertical posts and diagonal ties spanning two panels. It is the truss represented on Plate I.

Lock Nut. — See check nut.

Loop Eye. — An eye on the end of a rod or square bar, elongated into the form of a loop, as shown on Plate II., Figs. 4 and 8.

Lower Falsework. — The falsework below the level of the lower chords.

Main Diagonal. — A tension member of a truss, sloping upward towards the nearer end of the span. Main diagonals in iron bridges are not adjustable.

Moment. — The product of a force by its lever arm.

Moment of Inertia. — Represented by the equation, $I = Ap \quad = \Sigma r^2 dA$, where A is the area of the section considered, p the radius of gyration, and r the distance of any point from an assumed line lying either in the surface or outside of it: in other words, the moment of inertia of a surface about any axis is the product of the area by the square of the radius of gyration: or it is the summation of the products of each differential of the area by the square of its distance from the axis. If the axis lie in the surface, the moment of inertia is called a surface moment of inertia; while, if the axis be perpendicular to the surface, the moment of inertia is called a polar moment of inertia.

Monkey Wrench. — A wrench capable of being adjusted so as to fit nuts of different sizes. (Plate XII., Fig. 9.)

Moving Load. — See live load.

Mud-Sill. — A timber, usually from 6″ by 6″ to 12″ by 12″, at the bottom of a bent. It is laid horizontally in a trench, and the posts of the bent rest upon it.

Name Plate. — A plate of iron placed in a conspicuous position on a bridge, containing the name of the maker or designer of the structure.

Negative Rotation. — Rotation in a direction opposite to that of the hands of a watch.

Net Section. — See effective area.

Neutral Surface. — That part of a member subjected to bending, which is neither extended nor compressed. In symmetrical wrought-iron beams, with equal or nearly equal flanges, it is taken to be at the centre line of the web.

Nut. — A small piece of iron with a threaded core to fit on the screw end of a bolt, rod, or bar. (Plate II., Fig. 7.)

Order Bill. — A form of bill used in ordering material from the manufacturers.

Ornamental work. — Fancy work at the portals of a bridge to give it architectural effect. (Plates I. and IX.)

Overhead Bracing. — The upper lateral or sway bracing in through bridges. The term is usually applied to the vertical sway bracing, if there be any; if not, to the upper lateral bracing.

Packing. — See chord packing.

Panel. — That portion of a truss between adjacent posts or struts in Pratt truss bridges; called also a bay.

Panel Length. — The distance between two adjacent panel points of the same chord.

Panel Point. — See apex.

Pedestal. — The foot of a batter brace or end post. (Plate II., Fig. 12.)

Permanent Set. — The alteration in length of a piece of material which has been subjected to stress, remaining after the stress has been removed.

Pillar. — See column.

Pilot Nut, or Pin Pilot. — A nut, one end of which is a truncated cone, used to protect the

thread on the end of a pin when the latter is being driven into place. (Plate II., Fig. 8.)

Pin. — A cylindrical piece of iron used to connect bridge members. (Plate II., Fig. 8.)

Pitch. — The distance between centres of consecutive rivets of the same row.

Plane of Symmetry. — A plane dividing a body into two equal and symmetrical parts similarly disposed in reference to the plane.

Plant. — Tools and apparatus used in construction.

Plate. — A piece of flat iron wider than a bar. The common distinction between the two is that a plate is attached to something else, and acts with it, while a bar is an independent member.

Plate Girder. — A beam, built of plates and angles, used to span a small opening, generally less than sixty feet.

Pony Truss. — A truss so shallow as not to permit the use of overhead bracing.

Portal. — The space between the batter braces at one end of a bridge. Sometimes the term is applied to the portal bracing, though incorrectly.

Portal Bracing. — The combination of struts and ties in the plane of the batter braces at a portal, which transfers the wind pressure from the upper lateral system to the abutment or pier.

Portal Strut. — A strut belonging to the portal bracing. (Plate I.)

Positive Rotation. — Rotation in the direction of the hands of a watch.

Post. — A vertical strut. (Plate I.)

Pratt Truss. (called also the "Murphy-Whipple," or "Quadrangular" truss). — A single-intersection truss with vertical struts and diagonal ties.

Quadrangular Truss. — See Pratt truss.

Rachet Drill. — A hand machine for drilling rivet holes. (Plate XII., Fig. 4.)

Radius of Gyration. — The radius of gyration of any surface in reference to an axis is the distance from the axis to that point of the surface in which, if the whole area were concentrated, the moment of inertia in reference to the axis would be unchanged. It is therefore equal to the square root of the ratio of the moment of inertia over the area.

Ream. — To enlarge a rivet hole.

Reamer. — A tool for enlarging rivet holes. (Plate XII., Fig. 13.)

Re-enforcing Plate. — A plate used for the purpose of providing additional pin bearing, or strength to compensate for material cut away. (Plate II., Fig. 10.)

Re-railing Apparatus. — A contrivance for returning to the track a derailed vehicle. (Plate V.)

Resolve. — To divide a force into its component parts.

Ri. — A Japanese mile equal to about 2 ½ English miles.

Rivet. — A short piece of round iron tightly connecting two or more thicknesses of metal, and having, when in place, a head at each end.

Roadway. — The passage-way of a bridge for vehicles; usually means clear roadway, *q. v.*

Rod. — A piece of round iron.

Rolled Beam. — An I-beam. (Plate II., Fig. 2.)

Roller. — See expansion roller.

Roller Frame. — A light frame of iron for holding the rollers in position (Plate II., Fig. 16.)

Roller Plate. — The plate upon which the rollers rest, and which itself rests upon the masonry (Plate II., Fig. 12.)

Rope Sling. — See sling.

Run. — A line, or string; as, a run of joists.

Set. — The extension or compression of a piece of material under stress.

Shear, or Shearing-Stress. — The resistance which a body offers to the passage, or the tendency to passage, of one section along the next consecutive section.

Shim. — A filling piece. Here applied to a timber between the track-stingers and ties. (Plate I and Plate II., Fig. 14.)

Shipping-Bill. — A list of portions of a bridge, arranged in a manner to facilitate counting and checking when the material is received after shipment.

Shoe. — Another term for pedestal, *q. v.*

Shoe Pin Supporting Piece. — Intermediate bearing for a shoe pin. (Plate II., Fig. 12.)

Shoe Plate. — The plate on the under side of the shoe, resting on the rollers, bed-plate, or masonry. (Plate II., Fig. 12.)

Side Bracing. — A bracing for pony trusses to attach the panels of the top chord to the floor beams prolonged, in order to fix the panel points of the top chord. (Plate IX., Figs. 16 and 17.)

Single Intersection. — The style of truss in which the diagonals do not cross the posts. It is represented in skeleton on Plate XIII.

Skeleton Drawing. — A drawing which shows only the centre lines of members, such as a diagram of stresses. (Plate XIII.)

Skew Bridge. — A bridge in which the horizontal lines joining corresponding panel points of the opposite trusses are oblique to the planes of the trusses.

Sledge. — A heavy hammer, or mallet. (Plate XII., Fig. 6.)

Sleeve Nut. — An elongated nut, the core at one end having a right-hand thread, and that at the other a left-hand thread. Its office is to lengthen or shorten a tension member. (Plate II., Fig. 6.

Sling. — A loop of rope, very useful in erection for making a hasty attachment.

Slope. — Inclination to a horizontal plane.

Snatch Block. — A block with one side capable of being opened for the insertion of the rope. Its office is to change the direction of the rope. (Plate XII., Fig. 3.)

Span. — The length of a bridge from centre to centre of end pins or bearings.

Spikes. — Large nails for timber work.

Splay. — To spread at one end the two main portions of a member.

Splice. — A joint connected by means of plates.

Splice Plate. — A connecting plate at a joint. (Plate IX., Fig. 7.)

Spread. — The distance apart laterally.

Staggered Rivets. — Rivets are said to be staggered when each rivet of one row is opposite to the middle of the space between two rivets of the next row.

Static Load. — Dead load, *q. v.*

Stay Plate. — A plate always used at the end of a system of lacing or latticing. (Plate II., Figs. 9 and 11.)

Stiffening-Angle. — An angle iron used to stiffen the web of a beam. (Plate II., Fig. 10.)

Stiffener. — A piece of iron used to stiffen the web of a beam: it may be of angle or tee section. (Plate II., Fig. 10.)

Strain. — The extension or compression of a piece of material which is or has been under stress.

Stress. — The internal resisting force of a piece of material which is strained.

Stringer. — A beam to support the track and its load between panel points. (Plate I and Plate II., Fig. 10.)

Stringer Bracing Frames. — See bracing frames.

Stringer Support or Shelf. — A shelf of bent plate or angle iron rivetted to a floor beam for the purpose of helping to support a track stringer.

Strut. — A member which resists compression.

Sub-Punching. — The punching of rivet holes which have to be afterwards enlarged by reaming.

Sway Bracing. — Bracing transverse to the planes of the trusses. Its objects are to resist wind pressure, and to prevent undue vibration from passing loads. (Plate I.)

Table of Data. — A list of the known circumstances that affect the designing of a structure.

Tap. — A screw for cutting a thread in a nut.

Tee or T iron. — A piece of rolled iron of the section shown on Plate II., Fig. 4.

Tension. — A stress tending to elongate a body.

Thermal Stress. — A stress caused by variation in temperature.

Thread. — The spiral part of a screw or nut.

Through Bridge. — A bridge with overhead bracing.

Tie. — A tension member; generally refers to a main truss. A sleeper.

Timber Truck. — A small, strong wooden frame, with an iron roller set entirely below the upper surface. It is used in bridge erection for moving large timbers and heavy weights along a runway. (Plate XII., Fig. 7.)

Tongs. — Part of a riveting outfit; used for holding and carrying heated rivets. (Plate XII., Fig. 18.)

Track Stringer. — See stringer.

Track Tie. — A sleeper.

Transverse Component. — A component in a transverse direction; generally intended for a component perpendicular to the planes of the trusses.

Truss. — An assemblage of tension and compression members so arranged as to transmit loads from intermediate points to the ends.

Trussing. — A poor substitute for lacing or latticing. (Plate II., Fig. 13.)

Turn Buckle. — Similar to a sleeve nut, and for the same purpose. The sides are open, so that a crowbar may be inserted for the purpose of screwing up. Turn buckles are used for larger bars or rods than are sleeve nuts. (Plate II., Fig. 5.)

Ultimate Strength. — The greatest load that a portion of material can bear.

Uniform Load. — A load so distributed over an entire structure, that equal lengths everywhere receive equal portions.

U-nut. — A piece of iron, in the shape of the letter U, through which passes the threaded end of a rod, and which affords a bearing for the nut, with room to screw up the latter. Its use is not permissible in first-class bridge construction.

Upper Falsework. — The falsework that lies above the level of the lower chords.

Upset End. — An end of a rod or bar enlarged for the cutting thereon of a screw-thread.

Verbatim. — Word for word.

Vibration Rod. — A tension member for vertical or portal sway bracing. (Plate I.)

Washer. — A piece of cast or wrought iron to distribute the pressure of a bolt-head or nut over timber. (Plate II., Fig. 7.)

Web. — The portion of a truss or beam between the flanges. Its office is principally to resist shear. (Plate II., Fig. 15.)

Welded Heads. — Heads first worked into shape, then welded on the bars.

Whipple Truss. — See Linville truss.

Wind Shakes. — Cracks in timber caused by the wind while the tree was living.

Working-Drawings. — Drawings containing all the measurements necessary for construction.

Working-Stress. — The stress, usually the greatest stress, to which a piece of material is or should be subjected. Sometimes incorrectly employed for intensity of working-stress.

Wrench. — A tool for screwing up nuts. (Plate XII., Fig. 15.)

Yen. — A Japanese paper dollar of fluctuating value

INDEX.*

* No references are here made to the Glossary or List of Members.

明治十八年三月廿三日出版版權屆
同　　年六月廿七日出版
不販賣

MEMOIRS

OF THE

TÔKIÔ DAIGAKU

(UNIVERSITY OF TÔKIÔ)

No. 11.

A SYSTEM OF IRON RAILROAD BRIDGES FOR JAPAN

BY

J. A. L. WADDELL, C.E., B.A.Sc., MA.E.,

PROFESSOR OF CIVIL ENGINEERING IN THE UNIVERSITY OF TOKIO, JAPAN: CONSULTING ENGINEER FOR THE FIRM OF RAYMOND AND CAMPBELL, BRIDGE-BUILDERS OF COUNCIL BLUFFS, IOWA: MEMBER OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, RENSSELAER SOCIETY OF ENGINEERS, ENGINEERS' CLUB OF PHILADELPHIA, AND WESTERN SOCIETY OF ENGINEERS; ASSOCIATE MEMBER OF THE INSTITUTION OF CIVIL ENGINEERS, LONDON: AUTHOR OF THE DESIGNING OF ORDINARY IRON HIGHWAY BRIDGES.

(Tables and Plates)

PUBLISHED BY TÔKIÔ DAIGAKU

TÔKIÔ:

2545 (JAPANESE ERA)

1885 A. D.

Printed by "SEISHI-BUNSHA," Tokio Japan.

LIST OF TABLES.

[illegible]	[illegible]	[illegible]		30	[illegible]	[illegible]	[illegible]		30	[illegible]	[illegible]	[illegible]
.8802	1.8546	2.1070		40	.9492	3.0178	3.1792		40	.9894	6.8269	6.8998
.8816	1.8676	2.1185		50	.9502	3.0475	3.2074		50	.9899	6.9682	7.0396
.8829	1.8807	2.1301	**72**	00	.9511	3.0777	3.2361	**82**	00	.9903	7.1154	7.1853
.8843	1.8940	2.1418		10	.9520	3.1084	3.2653		10	.9907	7.2687	7.3372
.8857	1.9074	2.1537		20	.9528	3.1397	3.2951		20	.9911	7.4287	7.4957
.8870	1.9210	2.1657		30	.9537	3.1716	3.3255		30	.9914	7.5958	7.6613
.8884	1.9347	2.1786		40	.9546	3.2041	3.3565		40	.9918	7.7704	7.8344
.8897	1.9486	2.1902		50	.9555	3.2371	3.3881		50	.9922	7.9530	8.0156
.8910	1.9626	2.2027	**73**	00	.9563	3.2709	3.4203	**83**	00	.9925	8.1443	8.2055
.8923	1.9768	2.2153		10	.9572	3.3052	3.4532		10	.9929	8.3450	8.4047
.8936	1.9912	2.2282		20	.9580	3.3402	3.4867		20	.9932	8.5555	8.6138
.8949	2.0057	2.2412		30	.9588	3.3759	3.5209		30	.9936	8.7769	8.8337
.8962	2.0204	2.2543		40	.9596	3.4124	3.5559		40	.9939	9.0098	9.0652
.8975	2.0353	2.2677		50	.9605	3.4495	3.5915		50	.9942	9.2553	9.3092
.8988	2.0503	2.2812	**74**	00	.9613	3.4874	3.6280	**84**	00	.9945	9.5144	9.5668
.9001	2.0655	2.2949		10	.9621	3.5201	3.6652		10	.9948	9.7882	9.8391
.9013	2.0809	2.3088		20	.9628	3.5656	3.7032		20	.9951	10.0780	10.1275
.9026	2.0965	2.3228		30	.9636	3.6059	3.7420		30	.9954	10.3854	10.4334
.9038	2.1123	2.3371		40	.9644	3.6470	3.7817		40	.9957	10.7119	10.7585
.9051	2.1283	2.3515		50	.9652	3.6891	3.8222		50	.9959	11.0594	11.1045
.9063	2.1445	2.3662	**75**	00	.9659	3.7321	3.8637	**85**	00	.9962	11.430	11.474
.9075	2.1609	2.3811		10	.9667	3.7760	3.9061		10	.9964	11.826	11.868
.9088	2.1775	2.3961		20	.9674	3.8208	3.9495		20	.9967	12.251	12.291
.9100	2.1943	2.4114		30	.9681	3.8667	3.9939		30	.9969	12.706	12.745
.9112	2.2113	2.4269		40	.9689	3.9136	4.0394		40	.9971	13.197	13.235
.9124	2.2286	2.4426		50	.9696	3.9617	4.0859		50	.9974	13.727	13.763
.9135	2.2460	2.4586	**76**	00	.9703	4.0108	4.1336	**86**	00	.9976	14.301	14.336
.9147	2.2637	2.4748		10	.9710	4.0611	4.1824		10	.9978	14.924	14.958
.9159	2.2817	2.4912		20	.9717	4.1126	4.2324		20	.9980	15.605	15.637
.9171	2.2998	2.5078		30	.9724	4.1653	4.2837		30	.9981	16.350	16.38
.9182	2.3183	2.5247		40	.9730	4.2193	4.3362		40	.9983	17.169	17.19
.9194	2.3369	2.5419		50	.9737	4.2747	4.3901		50	.9985	18.075	18.103
.9205	2.3559	2.5593	**77**	00	.9744	4.3315	4.4454	**87**	00	.9986	19.081	19.107
.9216	2.3750	2.5770		10	.9750	4.3897	4.5022		10	.9988	20.206	20.230
.9228	2.3945	2.5949		20	.9757	4.4494	4.5604		20	.9989	21.470	21.494
.9239	2.4141	2.6131		30	.9763	4.5107	4.6202		30	.9990	22.904	22.926
.9250	2.4342	2.6316		40	.9769	4.5736	4.6817		40	.9992	24.542	24.562
.9261	2.4545	2.6504		50	.9775	4.0382	4.7448		50	.9993	26.432	26.451
.9272	2.4751	2.6695	**78**	00	.9781	4.7046	4.8097	**88**	00	.9994	28.636	28.654
.9283	2.4960	2.6888		10	.9787	4.7729	4.8765		10	.9995	31.242	31.258
.9293	2.5172	2.7085		20	.9793	4.8430	4.9452		20	.9996	34.368	34.382
.9304	2.5386	2.7285		30	.9799	4.9152	5.0159		30	.9997	38.188	38.202
.9315	2.5605	2.7488		40	.9805	4.9894	5.0886		40	.9997	42.964	42.976
.9325	2.5826	2.7695		50	.9811	5.0658	5.1630		50	.9998	49.104	49.114
.9336	2.6051	2.7904	**79**	00	.9816	5.1446	5.2408	**89**	00	.9998	57.290	57.299
.9346	2.6279	2.8117		10	.9822	5.2252	5.3205		10	.9999	68.750	68.757
.9356	2.6511	2.8334		20	.9827	5.3093	5.4026		20	.9999	85.940	85.946
.9367	2.6746	2.8555		30	.9833	5.3955	5.4874		30	1.0000	114.589	114.593
.9377	2.6985	2.8779		40	.9838	5.4845	5.5749		40	1.0000	171.885	171.888
.9387	9.7228	2.9006		50	.9843	5.5764	5.6653		50	1.0000	343.774	343.775

[illegible]	[illegible]	[illegible]		30	[illegible]	[illegible]	[illegible]		30	[illegible]	[illegible]	[illegible]
.8802	1.8546	2.1070		40	.9492	3.0178	3.1792		40	.9894	6.8269	6.8998
.8816	1.8676	2.1185		50	.9502	3.0475	3.2074		50	.9899	6.9682	7.0396
.8829	1.8807	2.1301	**72**	00	.9511	3.0777	3.2361	**82**	00	.9903	7.1154	7.1853
.8843	1.8940	2.1418		10	.9520	3.1084	3.2653		10	.9907	7.2687	7.3372
.8857	1.9074	2.1537		20	.9528	3.1397	3.2951		20	.9911	7.4287	7.4957
.8870	1.9210	2.1657		30	.9537	3.1716	3.3255		30	.9914	7.5958	7.6613
.8884	1.9347	2.1786		40	.9546	3.2041	3.3565		40	.9918	7.7704	7.8344
.8897	1.9486	2.1902		50	.9555	3.2371	3.3881		50	.9922	7.9530	8.0150
.8910	1.9626	2.2027	**73**	00	.9563	3.2709	3.4203	**83**	00	.9925	8.1443	8.2055
.8923	1.9768	2.2153		10	.9572	3.3052	3.4532		10	.9929	8.3450	8.4047
.8936	1.9912	2.2282		20	.9580	3.3402	3.4867		20	.9932	8.5555	8.6138
.8949	2.0057	2.2412		30	.9588	3.3759	3.5209		30	.9936	8.7769	8.8337
.8962	2.0204	2.2543		40	.9596	3.4124	3.5559		40	.9939	9.0098	9.0652
.8975	2.0353	2.2677		50	.9605	3.4495	3.5915		50	.9942	9.2553	9.3092
.8988	2.0503	2.2812	**74**	00	.9613	3.4874	3.6280	**84**	00	.9945	9.5144	9.5668
.9001	2.0655	2.2949		10	.9621	3.5261	3.6652		10	.9948	9.7882	9.8391
.9013	2.0809	2.3088		20	.9628	3.5656	3.7032		20	.9951	10.0780	10.1275
.9026	2.0965	2.3228		30	.9636	3.6059	3.7420		30	.9954	10.3854	10.4334
.9038	2.1123	2.3371		40	.9644	3.6470	3.7817		40	.9957	10.7119	10.7585
.9051	2.1283	2.3515		50	.9652	3.6891	3.8222		50	.9959	11.0594	11.1045
.9063	2.1445	2.3662	**75**	00	.9659	3.7321	3.8637	**85**	00	.9962	11.430	11.474
.9075	2.1609	2.3811		10	.9667	3.7760	3.9061		10	.9964	11.826	11.868
.9088	2.1775	2.3961		20	.9674	3.8208	3.9495		20	.9967	12.251	12.291
.9100	2.1943	2.4114		30	.9681	3.8667	3.9939		30	.9969	12.706	12.745
.9112	2.2113	2.4269		40	.9689	3.9136	4.0394		40	.9971	13.197	13.235
.9124	2.2286	2.4426		50	.9696	3.9617	4.0859		50	.9974	13.727	13.763
.9135	2.2460	2.4586	**76**	00	.9703	4.0108	4.1336	**86**	00	.9976	14.301	14.336
.9147	2.2637	2.4748		10	.9710	4.0611	4.1824		10	.9978	14.924	14.958
.9159	2.2817	2.4912		20	.9717	4.1126	4.2324		20	.9980	15.605	15.637
.9171	2.2998	2.5078		30	.9724	4.1653	4.2837		30	.9981	16.350	16.380
.9182	2.3183	2.5247		40	.9730	4.2193	4.3362		40	.9983	17.169	17.198
.9194	2.3369	2.5419		50	.9737	4.2747	4.3901		50	.9985	18.075	18.103
.9205	2.3559	2.5593	**77**	00	.9744	4.3315	4.4454	**87**	00	.9986	19.081	19.107
.9216	2.3750	2.5770		10	.9750	4.3897	4.5022		10	.9988	20.206	20.230
.9228	2.3945	2.5949		20	.9757	4.4494	4.5604		20	.9989	21.470	21.494
.9239	2.4141	2.6131		30	.9763	4.5107	4.6202		30	.9990	22.904	22.926
.9250	2.4342	2.6316		40	.9769	4.5736	4.6817		40	.9992	24.542	24.562
.9261	2.4545	2.6504		50	.9775	4.6382	4.7448		50	.9993	26.432	26.451
.9272	2.4751	2.6695	**78**	00	.9781	4.7046	4.8097	**88**	00	.9994	28.636	28.654
.9283	2.4960	2.6888		10	.9787	4.7729	4.8765		10	.9995	31.242	31.258
.9293	2.5172	2.7085		20	.9793	4.8430	4.9452		20	.9996	34.368	34.382
.9304	2.5386	2.7285		30	.9799	4.9152	5.0159		30	.9997	38.188	38.202
.9315	2.5605	2.7488		40	.9805	4.9894	5.0886		40	.9997	42.964	42.976
.9325	2.5826	2.7695		50	.9811	5.0658	5.1636		50	.9998	49.104	49.114
.9336	2.6051	2.7904	**79**	00	.9816	5.1446	5.2408	**89**	00	.9998	57.290	57.299
.9346	2.6279	2.8117		10	.9822	5.2252	5.3205		10	.9999	68.750	68.757
.9356	2.6511	2.8334		20	.9827	5.3093	5.4026		20	.9999	85.940	85.946
.9367	2.6746	2.8555		30	.9833	5.3955	5.4874		30	1.0000	114.589	114.593
.9377	2.6985	2.8779		40	.9838	5.4845	5.5749		40	1.0000	171.885	171.888
.9387	9.7228	2.9006		50	.9843	5.5764	5.6653		50	1.0000	343.774	343.775

[III.

STRERSECTION TRUSSES.

Member.	5 Pane W	5 Pane W1	8 Panel. W	8 Panel. W1	8 Panel. E	9 Panel. W	9 Panel. W1	9 Panel. E	Multiply by
Top Chord. 1	3	3	6	6	$\frac{22}{8}$	7	7	$\frac{28}{9}$	tan. θ
" " 2	3	3	$7\frac{1}{2}$	$7\frac{1}{2}$	$\frac{27}{8}$	9	9	$\frac{33}{9}$	
" " 3			8	8	$\frac{32}{8}$	10	10	$\frac{45}{9}$	
" " 4						10	10	$\frac{45}{9}$	
Bot. Chord. 1	2	2	$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{15}{8}$	4	4	$\frac{17}{9}$	
" " 2	2	2	$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{15}{8}$	4	4	$\frac{17}{9}$	
" " 3	3	3	6	6	$\frac{22}{8}$	7	7	$\frac{28}{9}$	
" " 4			$7\frac{1}{2}$	$7\frac{1}{2}$	$\frac{27}{8}$	9	9	$\frac{33}{9}$	
" " 5						10	10	$\frac{45}{9}$	
Batter Brace.	$\frac{10}{5}$	2	$\frac{28}{8}$	$3\frac{1}{2}$	$\frac{15}{8}$	$\frac{36}{9}$	4	$\frac{17}{9}$	Sec. θ
Diagonal. 1	$\frac{6}{5}$	1	$\frac{21}{8}$	$2\frac{1}{2}$	$\frac{11}{8}$	$\frac{25}{9}$	3	$\frac{18}{9}$	
" 2	$\frac{3}{5}$	0	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{5}{8}$	$\frac{21}{9}$	2	$\frac{11}{9}$	
" 3	$\frac{1}{5}$	-1	$\frac{10}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	$\frac{15}{9}$	1	$\frac{3}{9}$	
" 4			$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{3}{8}$	$\frac{10}{9}$	0	$\frac{7}{9}$	
" 5			$\frac{3}{8}$	$-1\frac{1}{2}$	$\frac{3}{8}$	$\frac{6}{9}$	-1	$\frac{6}{9}$	
Post (Thro. B.) 1	$\frac{4}{5}$	0	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{3}{8}$	$\frac{21}{9}$	2	$\frac{11}{9}$	To the stress on each post must be added *W.!*
" " 2			$\frac{10}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	$\frac{15}{9}$	1	$\frac{3}{9}$	
" " 3			$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{3}{8}$	$\frac{10}{9}$	0	$\frac{7}{9}$	
Post (Deck B.) 1	$\frac{6}{5}$	0	$\frac{21}{8}$	$1\frac{1}{2}$	$\frac{11}{8}$	$\frac{25}{9}$	2	$\frac{18}{9}$	
" " 2			$\frac{15}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{21}{9}$	1	$\frac{11}{9}$	
" " 3			$\frac{10}{8}$	$-\frac{1}{2}$	$\frac{7}{8}$	$\frac{15}{9}$	0	$\frac{3}{9}$	

USSES.

12 Panel.			13 Panel.			Multiply by
W	W_1	E	W	W_1	E	
2	12	$\frac{48}{12}$	$\frac{172}{13}$	$\frac{172}{13}$	$\frac{54}{13}$	Tan. α.
5	15	$\frac{59}{12}$	$\frac{218}{13}$	$\frac{218}{13}$	$\frac{66}{13}$	
7	17	$\frac{67}{12}$	$\frac{250}{13}$	$\frac{250}{13}$	$\frac{76}{13}$	
8	18	$\frac{71}{12}$	$\frac{270}{13}$	$\frac{270}{13}$	$\frac{82}{13}$	
8	18	$\frac{71}{12}$	$\frac{276}{13}$	$\frac{276}{13}$	$\frac{84}{13}$	
			$\frac{276}{13}$	$\frac{276}{13}$	$\frac{84}{13}$	
$\frac{1}{2}$	$5\frac{1}{2}$	$\frac{21}{12}$	$\frac{78}{13}$	$\frac{78}{13}$	$\frac{23}{13}$	
$\frac{1}{2}$	$5\frac{1}{2}$	$\frac{21}{12}$	$\frac{78}{13}$	$\frac{78}{13}$	$\frac{23}{13}$	
3	8	$\frac{36}{12}$	$\frac{114}{13}$	$\frac{114}{13}$	$\frac{40}{13}$	
2	12	$\frac{48}{12}$	$\frac{173}{13}$	$\frac{172}{13}$	$\frac{54}{13}$	
5	15	$\frac{56}{12}$	$\frac{218}{13}$	$\frac{218}{13}$	$\frac{64}{13}$	
7	17	$\frac{60}{12}$	$\frac{250}{13}$	$\frac{250}{13}$	$\frac{70}{13}$	
			$\frac{264}{13}$	$\frac{264}{13}$	$\frac{70}{13}$	
$\frac{6}{2}$	$5\frac{1}{2}$	$\frac{21}{12}$	$\frac{78}{13}$	$\frac{78}{13}$	$\frac{23}{13}$	Sec. α.
$\frac{0}{2}$	$2\frac{1}{2}$	$\frac{18}{12}$	$\frac{36}{13}$	$\frac{36}{13}$	$\frac{20}{13}$	
5	2	16	30	29	18	

11	12	1	12	26	1 2	13
	$\frac{21}{12}$	$\frac{1}{2}$	$\frac{11}{12}$	$\frac{55}{26}$	$\frac{1}{2}$	$\frac{13}{13}$

TABLE VI.

WORKING TENSILE STRESSES AND INITIAL TENSIONS

FOR

ADJUSTABLE ROUND AND SQUARE BARS.

Dia.	Intensity of Working stress = 4 tons.		Intensity of Working stress = 7.5 tons		Initial Tensions.		Dia.
	●	■	●	■	●	■	
$\frac{3}{4}$"	1.268	1.517	2.815	3.574	0.500	0.635	$\frac{3}{4}$"
$\frac{13}{16}$"	1.451	1.846	3.261	4.157	0.625	0.794	$\frac{13}{16}$"
$\frac{7}{8}$"	1.650	2.111	3.760	4.789	0.750	0.953	$\frac{7}{8}$"
$\frac{15}{16}$"	1.885	2.405	4.303	5.481	0.875	1.111	$\frac{15}{16}$"
1"	2.140	2.730	4.890	6.230	1.000	1.270	1"
$1\frac{1}{16}$"	2.423	3.087	5.525	7.038	1.125	1.429	$1\frac{1}{16}$"
$1\frac{1}{8}$"	2.726	3.476	6.205	7.904	1.250	1.588	$1\frac{1}{8}$"
$1\frac{3}{16}$"	3.057	3.894	6.931	8.830	1.375	1.746	$1\frac{3}{16}$"
$1\frac{1}{4}$"	3.408	4.347	7.704	9.814	1.500	1.905	$1\frac{1}{4}$"
$1\frac{5}{16}$"	3.787	4.828	8.523	10.856	1.625	2.064	$1\frac{5}{16}$"
$1\frac{3}{8}$"	4.190	5.341	9.387	11.956	1.750	2.223	$1\frac{3}{8}$"
$1\frac{7}{16}$"	4.617	5.883	10.298	13.117	1.875	2.381	$1\frac{7}{16}$"
$1\frac{1}{2}$"	5.068	6.460	11.253	14.335	2.000	2.540	$1\frac{1}{2}$"
$1\frac{9}{16}$"	5.547	7.065	12.256	15.611	2.125	2.699	$1\frac{9}{16}$"
$1\frac{5}{8}$"	6.046	7.706	13.304	16.947	2.250	2.858	$1\frac{5}{8}$"
$1\frac{11}{16}$"	6.573	8.376	14.399	18.342	2.375	3.016	$1\frac{11}{16}$"
$1\frac{3}{4}$"	7.120	9.077	15.540	19.794	2.500	3.175	$1\frac{3}{4}$"
$1\frac{13}{16}$"	7.695	9.810	16.726	21.305	2.625	3.334	$1\frac{13}{16}$"
$1\frac{7}{8}$"	8.294	10.571	17.959	22.874	2.750	3.493	$1\frac{7}{8}$"
$1\frac{15}{16}$"	8.917	11.365	19.237	24.503	2.875	3.651	$1\frac{15}{16}$"
2"	9.568	12.190	20.562	26.190	3.000	3.810	2"
$2\frac{1}{16}$"	10.239	13.047	21.933	27.935	3.125	3.969	$2\frac{1}{16}$"
$2\frac{1}{8}$"	10.938	13.936	23.349	29.739	3.250	4.128	$2\frac{1}{8}$"
$2\frac{3}{16}$"	11.657	14.854	24.812	31.603	3.375	4.286	$2\frac{3}{16}$"
$2\frac{1}{4}$"	12.404	15.807	26.321	33.524	3.500	4.445	$2\frac{1}{4}$"
$2\frac{5}{16}$"	13.175	16.788	27.875	35.504	3.625	4.604	$2\frac{5}{16}$"
$2\frac{3}{8}$"	13.970	17.801	29.476	37.541	3.750	4.763	$2\frac{3}{8}$"
$2\frac{7}{16}$"	14.793	18.843	31.123	39.640	3.875	4.921	$2\frac{7}{16}$"
$2\frac{1}{2}$"	15.636	19.920	32.815	41.795	4.000	5.080	$2\frac{1}{2}$"

TABLE VII.

Sizes of Hip Verticals and Beam Hangers

For

Single Track Bridges.

Panel Length.	Hip Verticals.		Beam Hanger.
	S. R.	Size.	Size.
10′	2.96□″	$2-1\frac{1}{4}''\square$	$1''\square$ up.
11′	3.18□″	$2-1\frac{5}{16}''\square$	$1\frac{1}{16}''\square$ up.
12′	3.38□″	$2-1\frac{6}{16}''\square$	$1\frac{1}{16}''\square$ up.
13′	3.56□″	$2-1\frac{3}{8}''\square$	$1\frac{1}{8}''\square$ up.
14′	3.70□″	$2-1\frac{3}{8}''\square$	$1\frac{1}{8}''\square$ up.
15′	3.86□″	$2-1\frac{7}{16}''\square$	$1\frac{3}{16}''\square$ up.
16′	4.00□″	$2-1\frac{7}{16}''\square$	$1\frac{3}{16}''\square$ up.
17′	4.14□″	$2-1\frac{7}{16}''\square$	$1\frac{3}{16}''\square$ up.
18′	4.26□″	$2-1\frac{1}{2}''\square$	$1\frac{1}{4}''\square$ up.
19′	4.40□″	$2-1\frac{1}{2}''\square$	$1\frac{1}{4}''\square$ up.
20′	4.58□″	$2-1\frac{1}{4}''\times 2''$	$1\frac{1}{4}''\square$ up.
21′	4.74□″	$2-1\frac{1}{2}''\times 2''$	$1\frac{5}{16}''\square$ up.
22′	4.90□″	$2-1\frac{1}{4}''\times 2''$	$1\frac{5}{16}''\square$ up.
23′	5.04□″	$2-1\frac{1}{8}''\times 2\frac{1}{4}''$	$1\frac{5}{16}''\square$ up.
24′	5.20□″	$2-1\frac{1}{4}''\times 2\frac{1}{2}''$	$1\frac{3}{8}''\square$ up.

TABLE VIII.

INTENSITIES OF WORKING COMPRESSIVE STRESS

FOR

Channel Struts in Trusses.

Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙
10	4.205	4.140	3.990	28	3.142	2.826	2.477	46	2.241	1.792	1.419	64	1.569	1.130	0.832
10½	4.175	4.103	3.947	28½	3.114	2.792	2.441	46½	2.219	1.769	1.398	64½	1.553	1.116	[illegible]
11	4.145	4.066	3.904	29	3.086	2.759	2.404	47	2.198	1.746	1.377	65	1.538	1.102	0.809
11½	4.114	4.029	3.862	29½	3.059	2.725	2.368	47½	2.176	1.723	1.356	65½	1.523	1.088	0.797
12	4.085	3.993	3.819	30	3.031	2.692	2.332	48	2.155	1.701	1.335	66	1.508	1.075	0.786
12½	4.053	3.956	3.775	30½	3.005	2.659	2.297	48½	2.134	1.679	1.315	66½	1.493	1.062	0.775
13	4.023	3.919	3.732	31	2.976	2.627	2.262	49	2.113	1.658	1.296	67	1.479	1.049	0.765
13½	3.993	3.882	3.688	31½	2.950	2.595	2.227	49½	2.092	1.636	1.276	67½	1.464	1.036	0.754
14	3.962	3.845	3.645	32	2.923	2.563	2.193	50	2.072	1.615	1.256	68	1.450	1.024	0.744
14½	3.932	3.807	3.601	32½	2.897	2.531	2.160	50½	2.051	1.594	1.238	68½	1.435	1.011	0.734
15	3.901	3.770	3.557	33	2.870	2.500	2.127	51	2.031	1.574	1.219	69	1.421	0.999	0.724
15½	3.872	3.732	3.514	33½	2.844	2.469	2.094	51½	2.011	1.554	1.200	69½	1.407	0.987	0.714
16	3.841	3.695	3.470	34	2.818	2.438	2.061	52	1.991	1.534	1.182	70	1.393	0.975	0.704
16½	3.811	3.657	3.424	34½	2.791	2.408	2.030	52½	1.971	1.514	1.165	70½	1.379	0.963	0.694
17	3.781	3.620	3.382	35	2.766	2.378	1.999	53	1.952	1.495	1.148	71	1.366	0.951	0.685
17½	3.751	3.583	3.341	35½	2.740	2.348	1.968	53½	1.933	1.476	1.130	71½	1.353	0.939	0.676
18	3.721	3.546	3.295	36	2.715	2.318	1.937	54	1.914	1.457	1.113	72	1.340	0.928	0.667
18½	3.692	3.508	3.252	36½	2.689	2.289	1.908	54½	1.895	1.438	1.097	72½	1.327	0.917	0.658
19	3.662	3.471	3.209	37	2.664	2.260	1.878	55	1.876	1.420	1.081	73	1.314	0.906	0.649
19½	3.632	3.434	3.166	37½	2.639	2.231	1.849	55½	1.857	1.402	1.064	73½	1.301	0.895	0.640
20	3.602	3.397	3.123	38	2.614	2.203	1.820	56	1.839	1.384	1.048	74	1.288	0.885	0.632
20½	3.573	3.360	3.080	38½	2.589	2.175	1.793	56½	1.821	1.366	1.033	74½	1.275	0.874	0.624
21	3.543	3.323	3.038	39	2.565	2.147	1.765	57	1.803	1.349	1.018	75	1.263	0.864	0.616
21½	3.514	3.286	2.996	39½	2.540	2.119	1.738	57½	1.785	1.332	1.003	75½	1.251	0.854	0.608
22	3.485	3.250	2.953	40	2.516	2.092	1.710	58	1.768	1.315	0.988	76	1.238	0.844	0.600
22½	3.456	3.214	2.912	40½	2.492	2.065	1.684	58½	1.751	1.298	0.974	76½	1.226	0.834	0.592
23	3.426	3.178	2.871	41	2.469	2.039	1.658	59	1.734	1.282	0.960	77	1.214	0.824	0.584
23½	3.397	3.142	2.830	41½	2.445	2.013	1.633	59½	1.717	1.266	0.946	77½	1.202	0.814	0.576
24	3.369	3.106	2.790	42	2.422	1.987	1.607	60	1.700	1.250	0.933	78	1.191	0.805	0.569
24½	3.340	3.070	2.750	42½	2.399	1.962	1.583	60½	1.683	1.235	0.920	78½	1.179	0.795	0.562
25	3.311	3.035	2.710	43	2.375	1.937	1.558	61	1.666	1.219	0.907	79	1.168	0.786	0.555
25½	3.282	2.999	2.676	43½	2.352	1.912	1.533	61½	1.649	1.203	0.894	79½	1.157	0.777	[illegible]
26	3.254	2.964	2.630	44	2.329	1.887	1.509	62	1.632	1.188	0.881	80	1.146	0.768	0.541
26½	3.226	2.929	2.591	44½	2.307	1.863	1.486	62½	1.616	1.173	0.868	80½	1.135	0.759	0.534
27	3.198	2.895	2.553	45	2.285	1.839	1.464	63	1.600	1.159	0.856	81	1.124	0.750	[illegible]
27½	3.170	2.863	2.515	45½	2.263	1.813	1.442	63½	1.584	1.145	0.844	81½	1.114	0.741	[illegible]

TABLE IX.

Intensities of Working Compressive Stress for Channel Struts in Lateral Systems, Portal Bracing and Vertical Sway Bracing.

Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙	Ratio L to D	□ □	□ ⊙	⊙ ⊙
10	5.677	5.588	5.386	28	4.313	3.880	3.401	46	3.114	2.490	1.972	64	2.200	1.585	1.166
10½	5.640	5.542	5.332	28½	4.276	3.835	3.352	46½	3.084	2.458	1.943	64½	2.179	1.566	1.150
11	5.601	5.496	5.277	29	4.240	3.790	3.302	47	3.055	2.427	1.913	65	2.158	1.547	1.132
11½	5.563	5.448	5.222	29½	4.194	3.746	3.254	47½	3.026	2.397	1.885	65½	2.138	1.528	1.110
12	5.525	5.402	5.167	30	4.167	3.701	3.206	48	2.998	2.366	1.856	66	2.117	1.509	1.102
12½	5.486	5.354	5.111	30½	4.132	3.658	3.159	48½	2.969	2.337	1.829	66½	2.096	1.491	1.089
13	5.448	5.307	5.055	31	4.096	3.614	3.111	49	2.941	2.307	1.801	67	2.076	1.473	1.072
13½	5.410	5.260	4.999	31½	4.060	3.571	3.065	49½	2.913	2.278	1.775	67½	2.056	1.455	1.050
14	5.372	5.212	4.942	32	4.025	3.529	3.019	50	2.885	2.250	1.748	68	2.036	1.438	1.045
14½	5.333	5.164	4.885	32½	3.990	3.486	2.975	50½	2.858	2.221	1.723	68½	2.017	1.421	1.031
15	5.295	5.116	4.828	33	3.955	3.444	2.930	51	2.830	2.193	1.697	69	1.997	1.403	1.017
15½	5.252	5.068	4.771	33½	3.920	3.403	2.886	51½	2.803	2.166	1.672	69½	1.978	1.387	1.003
16	5.218	5.019	4.713	34	3.885	3.361	2.842	52	2.776	2.138	1.647	70	1.959	1.370	.990
16½	5.180	4.971	4.656	34½	3.851	3.321	2.800	52½	2.750	2.112	1.623	70½	1.940	1.354	.977
17	5.141	4.922	4.599	35	3.816	3.280	2.757	53	2.723	2.085	1.599	71	1.922	1.338	.964
17½	5.103	4.874	4.532	35½	3.782	3.240	2.716	53½	2.697	2.059	1.576	71½	1.903	1.322	.951
18	5.065	4.826	4.485	36	3.748	3.200	2.675	54	2.671	2.033	1.553	72	1.885	1.306	.938
18½	5.027	4.777	4.428	36½	3.715	3.161	2.635	54½	2.646	2.008	1.531	72½	1.867	1.291	.926
19	4.988	4.729	4.371	37	3.681	3.122	2.594	55	2.620	1.983	1.508	73	1.849	1.276	.914
19½	4.950	4.680	4.314	37½	3.648	3.084	2.555	55½	2.590	1.958	1.487	73½	1.832	1.261	.902
20	4.912	4.632	4.258	38	3.614	3.045	2.516	56	2.570	1.933	1.465	74	1.814	1.246	.890
20½	4.874	4.584	4.202	38½	3.582	3.008	2.479	56½	2.545	1.910	1.444	74½	1.797	1.231	.879
21	4.836	4.535	4.145	39	3.549	2.970	2.441	57	2.521	1.886	1.423	75	1.779	1.217	.867
21½	4.798	4.487	4.090	39½	3.516	2.933	2.404	57½	2.497	1.862	1.403	75½	1.763	1.203	.856
22	4.760	4.439	4.054	40	3.484	2.897	2.367	58	2.472	1.839	1.382	76	1.746	1.189	.845
22½	4.722	4.392	3.978	40½	3.452	2.861	2.332	58½	2.449	1.816	1.363	76½	1.729	1.175	.834
23	4.684	4.344	3.924	41	3.420	2.825	2.296	59	2.425	1.794	1.343	77	1.713	1.161	.823
23½	4.647	4.297	3.870	41½	3.388	2.790	2.261	59½	2.402	1.772	1.324	77½	1.696	1.148	.813
24	4.609	4.249	3.816	42	3.357	2.755	2.227	60	2.378	1.750	1.305	78	1.680	1.135	.802
24½	4.572	4.202	3.763	42½	3.326	2.720	2.194	60½	2.355	1.728	1.287	78½	1.664	1.122	.793
25	4.534	4.155	3.710	43	3.295	2.686	2.160	61	2.332	1.707	1.269	79	1.648	1.109	.783
25½	4.497	4.109	3.657	43½	3.264	2.652	2.128	61½	2.310	1.686	1.251	79½	1.633	1.096	.773
26	4.460	4.062	3.605	44	3.233	2.619	2.095	62	2.288	1.665	1.233	80	1.617	1.084	.763
26½	4.423	4.016	3.553	44½	3.203	2.586	2.064	62½	2.266	1.645	1.216	80½	1.602	1.071	.754
27	4.386	3.970	3.502	45	3.173	2.553	2.033	63	2.244	1.624	1.199	81	1.587	1.059	.744
27½	4.349	3.925	3.451	45½	3.143	2.522	2.002	63½	2.222	1.605	1.183	81½	1.572	1.047	.735

TABLE

APPROXIMATE WORKING LOADS F
IN LATERAL SYSTEMS OR VE

Length of Strut in feet.			7″—20# I		7″—18# I		6″—16# I		6″—13.5# I		5′
□□	□○	○○	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Sid way
1			26.7	27.0	24.2	24.5	21.5	21.7	18.2	18.2	16
2	1.5	1	26.4	26.9	24.0	24.4	21.2	21.6	17.7	18.1	15
4	3	2	25.0	26.7	22.7	24.3	20.0	21.5	16.5	18.0	13
6	4.5	3	23.0	26.5	21.0	24.2	18.0	21.2	15.0	17.9	11.
8	6	4	20.5	26.2	18.7	24.0	15.8	21.0	13.2	17.7	9.
10	7.5	5	18.2	26.0	16.5	23.7	13.7	20.7	11.5	17.5	8.
12	9	6	15.7	25.7	14.5	23.5	11.7	20.2	10.4	17.1	6.
14	10.5	7	13.7	25.2	12.5	23.1	10.0	19.7	9.2	16.7	5.
16	12	8	12.0	24.7	11.2	22.7	8.7	19.2	8.0	16.2	4.
18	13.5	9	10.5	24.2	9.7	22.2	7.5	18.7	6.7	15.7	3.
20	15	10	9.0	23.7	8.5	21.7	6.5	18.1	5.5	15.2	3.
22	16.5	11	8.0	23.0	7.5	21.2	5.7	17.5	5.0	14.7	2.
24	18	12	7.0	22.3	6.5	20.7	5.0	17.0	4.5	14.2	2.
26	19.5	13	6.2	21.7	5.7	20.2	4.2	16.2	4.0	13.7	2.
28	21	14	5.5	21.1	5.0	19.5	3.7	15.5	3.5	13.2	1.
30	22.5	15	5.0	20.5	4.5	18.7	3.5	15.0	3.1	12.7	1.
32	24	16	4.5	19.8	4.0	18.0	3.0	14.5	2.7	12.2	1.
34	25.5	17	4.0	19.2	3.5	17.5	2.7	14.0	2.5	11.7	1.
36	27	18	3.5	18.6	3.2	17.0	2.5	13.5	2.2	11.2	1.
38	28.5	19	3.2	18.0	3.0	16.5	2.2	13.0	2.0	10.7	1.
40	30.	20	3.0	17.4	2.7	16.0	2.0	12.5	1.7	10.2	0.

TABLE

APPROXIMATE WORKING LOADS F

IN LATERAL SYSTEMS OR VE

Length of Strut in feet.			7″—20# I		7″—18# I		6″—16# I		6″—13.5# I		5′
□□	□○	○○	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Side-ways.	Edge-ways.	Sid way
1			26.7	27.0	24.2	24.5	21.5	21.7	18.2	18.2	16
2	1.5	1	26.4	26.9	24.0	24.4	21.2	21.6	17.7	18.1	15
4	3	2	25.0	26.7	22.7	24.3	20.0	21.5	16.5	18.0	13
6	4.5	3	23.0	26.5	21.0	24.2	18.0	21.2	15.0	17.9	11.
8	6	4	20.5	26.2	18.7	24.0	15.8	21.0	13.2	17.7	9.
10	7.5	5	18.2	26.0	16.5	23.7	13.7	20.7	11.5	17.5	8.
12	9	6	15.7	25.7	14.5	23.5	11.7	20.2	10.4	17.1	6.
14	10.5	7	13.7	25.2	12.5	23.1	10.0	19.7	9.2	16.7	5.
16	12	8	12.0	24.7	11.2	22.7	8.7	19.2	8.0	16.2	4.
18	13.5	9	10.5	24.2	9.7	22.2	7.5	18.7	6.7	15.7	3.
20	15	10	9.0	23.7	8.5	21.7	6.5	18.1	5.5	15.2	3.
22	16.5	11	8.0	23.0	7.5	21.2	5.7	17.5	5.0	14.7	2.
24	18	12	7.0	22.3	6.5	20.7	5.0	17.0	4.5	14.2	2.
26	19.5	13	6.2	21.7	5.7	20.2	4.2	16.2	4.0	13.7	2.
28	21	14	5.5	21.1	5.0	19.5	3.7	15.5	3.5	13.2	1.
30	22.5	15	5.0	20.5	4.5	18.7	3.5	15.0	3.1	12.7	1.
32	24	16	4.5	19.8	4.0	18.0	3.0	14.5	2.7	12.2	1.
34	25.5	17	4.0	19.2	3.5	17.5	2.7	14.0	2.5	11.7	1.
36	27	18	3.5	18.6	3.2	17.0	2.5	13.5	2.2	11.2	1.
38	28.5	19	3.2	18.0	3.0	16.5	2.2	13.0	2.0	10.7	1.
40	30.	20	3.0	17.4	2.7	16.0	2.0	12.5	1.7	10.2	0.

TABLE XI.

DIMENSIONS AND WEIGHTS

OF

Track Stringers.

Panel Length.	Track Stringers.			
	Web	Up. Fl.	L. Fl.	Weight of two Stringers and their Bracing.
10′	15″	50 •	I	1100
11′	15″	50 •	I	1200
12′	15″	50 •	I	1300
13′	15″	67 •	I	1842
14′	15″	67 •	I	1976
15′	15″	80 •	I	2500
16′	⅜″ × 24″	2—3″ × 3½″—10.4• L	2—3″ × 3½″—13.1• L	3400
17′	⅜″ × 25″	2—3″ × 3 ″—10.4• L	2—3″ × 3½″—9.0• L 1—⅜″ × 7½″—Pl.	3570
18′	⅜″ × 26″	2—3″ × 3½″—11.7• L	2—3″ × [illegible]½″—10.4• L 1—⅜″ × 7 ″Pl.	3860
19′	⅜″ × 27″	2—3½″ × 3½″—12.7• L	2—3½″ × 3½″—9.7• L 1—7/16″ × 7½″ Pl.	4240
20′	⅜″ × 28″	2—3½″ × 3½″—14.1• L	2—3½″ × 3½″—9.7• L 1—½″ × 7½″ Pl.	4630* 4750
21′	⅜″ × 29″	2—3″ × 3½″—14.4• L	2—3½″ × 4″—10.5• L 1—7/16″ × 6″ Pl.	5100* 5220
2[illegible]′	⅜″ × 3[illegible]″	2—3½″ × 4″—15.2• L	2—3½″ × 4″—10.5• L 1—½″ × 8″ Pl.	5490* 5610
23′	⅜″ × 31″	2—3 ″ × 3½″—15.6• L	2—3″ × 4″—12.2• L 1—½″ × [illegible]″ Pl.	6040
24	⅜″ × 32″	2—4″ × 4″—16.2• L	2—3½″ × 4″—12• L 1—½″ × [illegible]″ Pl.	6520

* The upper line to be used when wooden shims are employed, and the under line when not.

TABLE XII.

Dimensions and Weights of Floor Beams

for

Single Track Bridges.

Panel Length.	Floor Beams.			
	Web.	Up. Fl.	L. Fl.	Weight.
10′	⅜″ × 24″	2—4″ × 4″—16.2• L	2—3″ × 5″—17.9• L	2350
11′	⅜″ × 25″	2—4″ × 5″—16.4• L	2—3″ × 5″—17.9 L•	2400
12′	⅜″ × 26″	2—3″ × 4″—17.• L	2—4″ × 5″—18.3• L	2450
13′	⅜″ × 27″	2—3½″ × 5″—17.2• L	2—3½″ × 5″—19.• L	2520
14′	⅜″ × 28″	2—3½″ × 5″—17.2• L	2—3½″ × 5″—19• L	2580
15′	⅜″ × 30″	2—3½″ × 5″—17.2• L	2—3½″ × 5″—19• L	2600
16′	⅜″ × 32″	2—4″ × 5″—16.4• L	2—3″ × 5″—17.9• L	2620
17′	⅜″ × 33″	2—3½″ × 4″—16.7• L	2—4″ × 5″—18.3• L	2720
18′	⅜″ × 34″	2—3½″ × 4″—16.7• L	2—4″ × 5″—18.3• L	2770
		2—3½″ × 5″—19• L	2—3½ × 5″—13.7• L 1—7/16″ × 10″ Pl.	3000
19′	⅜″ × 35″	2—3″ × 4″—17• L	2—3[illegible]″ × 5″—19• L	2850
		2—3″ × 5″—19.5• L	2—3½″ × 5″—13.7• L 1—½″ × 10″ Pl.	3130
20′	⅜″ × 36″	2—4″ × 5″—18.3• L	2—3½″ × 5″—13.7• L 1—⅜″ × 10″ Pl.	2950 or 3200
		2—4″ × 5″—20.2• L	2—3½″ × 5″—15.5• L 1—⅜″ × 10″ Pl.	3280 or 3530
21′	⅜″ × 36″	2—3½″ × 5″—13.7• L 1—7/16″ × 10½″ Pl.	2—4″ × 5″—16.4• L 1—7/16″ × 10″ Pl.	3400 or 3650
22′	⅜″ × 36″	2—3½″ × 5″—15.5• L 1—7/16″ × 11″ Pl.	2—4″ × 5″—18.3• L 1—7/16″ × 10″ Pl.	3750 or 4000
23′	⅜″ × 36″	2—3½″ × 5″—15.5• L 1—½″ × 10½ Pl.	2—4″ × 5″—18.3• L 1—½″ × 10″ Pl.	4050
24′	⅜″ × 36″	2—3½″ × 5″—17.2• L 1—7/16″ × 10½″ Pl.	2—4″ × 5″—18.3• L 1—9/16″ × 10″ Pl.	4100.

N. B. These weights do not include those of the portions of pony truss floor beams projecting outside of the trusses. Where two floor beams are given, the light one is for spans below two hundred feet in length and the heavy one for spans above the same; and where two weights are given for the same beam the smaller is for the case of abutting stringers and the larger for stringers resting on floor beams.

'ERTICAL SWAY BRACING

Span	Pan. 4.	Pan. 5.	Pan. 6.	Pan. 7.
Pony Trusess	2—4"—6# [	2—4"—6# [		
	$1\frac{5}{16}$" ⊙	$1\frac{1}{8}$" ⊙		

XIV.

ENTS &c.

el Pins.

Dia.	Re… Iro… Trusses.	Resisting Moments for Bending. Iron. Trusses.	Iron. Lat. Syst.	Steel. Trusses.	Steel. Lat. Syst.	Resisting Shearing Stress. Steel. Trusses.
1 ¼ "		56.5	84.7	90.4	135.6	54.8
1 ⅜ "		61.7	92.5	98.7	148.1	58.1
1 ½ "		67.1	100.6	107.4	161.1	61.5
1 ⅝ "		72.8		116.5		65.0
1 ¾ "		78.9		126.2		68.5
1 ⅞ "		85.3		136.5		72.1
2 "	5.9	92.0		147.2		75.8
2 ⅛ "	7.1	99.1		158.6		79.7
2 ¼ "	8.4	106.5		170.4		83.7
2 ⅜ "	9.9	114.3		182.9		87.8
2 ½ "	11.5	122.5		196.0		91.9
2 ⅝ "	13.3	131.0		209.6		96.1
2 ¾ "	15.3	140.0		224.0		100.4
2 ⅞ "	17.5	149.3		238.9		104.8
3 "	19.9	159.0		254.4		109.3
3 ⅛ "	22.5	169.2		270.7		113.4
3 ¼ "	25.3	179.8		287.7		118.4
3 ⅜ "	28.3	190.8		305.3		123.2
3 ½ "	31.6	202.2		323.5		128.0
3 ⅝ "	35.1	214.1		342.6		133.0
3 ¾ "	38.8	226.5		362.4		138.0
3 ⅞ "	42.8	239.3		382.9		143.3
4 "	47.1	252.6		404.5		148.5
4 ⅛ "	51.7					

N IRON MILL'S CHAN

N SQUARE INCHES AES.

[. A.		9"[. B.			10"[. B.			12"[. C.			15"[. A.			Thickness of web in inches.
A	F	W	A	A	W	A	F	W	A	F	W	A	F	
														0.250
														0.275
		18.00	5.40	2.43										0.300
4.35	2.50	18.60	5.58	2.45	16.00	4.76	3.01							0.325
		19.35	5.81	2.48		.06	3.04							0.350
		20.10	6.03	2.50		.36	3.06							0.375
		20.85	6.26	2.53		.66	3.09							0.400
		21.60	6.48	2.55		.96	3.11							0.425
		22.35	6.71	2.58		.26	3.14	30.00	9.00	2.71				0.450
		23.10	6.93	2.60		.56	3.16	30.72	9.22	2.73				0.475
		23.85	7.16	2.63		.86	3.19	31.72	9.52	2.75				0.500
		24.60	7.38	2.65				32.72	9.82	2.78	40.00	12.00	3.53	0.525
		25.35	7.61	2.68				33.72	10.12	2.80	41.25	12.38	3.56	0.550
		26.10	7.83	2.70				34.72	10.42	2.83	42.50	12.75	3.58	0.575
		26.85	8.06	2.73				35.72	10.72	2.85	43.75	13.13	3.61	0.600
		27.60	8.28	2.75				36.72	11.02	2.88	45.00	13.50	3.63	0.625
		28.35	8.51	2.78				37.72	11.32	2.90	46.25	13.88	3.66	0.650
		29.10	8.73	2.80				38.72	11.62	2.93	47.50	14.25	3.68	0.675
		29.85	8.96	2.83				39.72	11.92	2.95	48.75	14.63	3.71	0.700
								40.72	12.22	2.98	50.00	15.00	3.73	0.725
								41.72	12.52	3.00	51.25	15.38	3.76	0.750
								42.72	12.82	3.03	52.50	15.75	3.78	0.775
								43.72	13.12	3.05	53.75	16.13	3.81	0.800
								44.72	13.42	3.08	55.00	16.50	3.83	0.825
								45.72	13.72	3.10	56.25	16.88	3.86	0.850
								46.72	14.02	3.13	57.50	17.25	3.88	0.875
								47.72	14.32	3.15	58.75	17.63	3.91	0.900
								48.72	14.62	3.18	60.00	18.00	3.93	0.925
								49.72	14.92	3.20				0.950

TABLE XVII.

Formula $p = 0.25 \sqrt{d}$, where p is the pressure in tons per lineal
oller in inches. The first and last vertical lines give the diameters,
engths of rollers. The intermediate spaces contain the permissible

7″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.
5.62	5.95	6.28	6.61	6.94	7.28	7.61	7.94	8.27	8.60	8.93	9.26	9.59	9.92	1¾″
5.82	6.16	6.50	6.85	7.19	7.53	7.87	8.22	8.56	8.90	9.24	9.58	9.93	10.27	1⅞″
6.01	6.36	6.72	7.07	7.43	7.80	8.13	8.49	8.84	9.19	9.55	9.90	10.25	10.61	2″
6.19	6.60	6.92	7.29	7.65	8.02	8.38	8.75	9.11	9.42	9.84	10.20	10.57	10.93	2⅛″
6.38	6.75	7.13	7.50	7.88	8.25	8.63	9.00	9.38	9.75	10 13	10.50	10.88	11.25	2¼″
6.55	6.94	7.32	7.71	8.09	8.48	8.86	9.25	9.63	10.02	10.40	10.79	11.17	11.56	2⅜″
6.72	7.12	7.51	7.91	8.30	8.70	9.09	9.49	9.88	10.28	10.67	11.07	11.46	11.86	2½″
6.89	7.29	7.70	8.10	8.51	8.91	9.32	9.72	10.13	10.53	10.94	11.34	11.75	12.15	2⅝″
7.05	7.46	7.88	8.29	8.71	9.12	9.54	9.95	10.37	10.78	11.19	11.61	12.02	12.44	2¾″
7.21	7.63	8.05	8.48	8.90	9.33	9.75	10.17	10.60	11.02	11.45	11.87	12.29	12.72	2⅞″
7.36	7.79	8.23	8.66	9.09	9.53	9.96	10.39	10.83	11.26	11.69	12.12	12.56	12.99	3″
7.51	7.95	8.40	8.84	9.28	9.72	10.16	10.61	11.05	11.50	11.92	12.37	12.82	13.26	3⅛″
7.66	8.11	8.56	9.01	9.46	9.92	10.37	10.82	11.27	11.72	12.17	12.62	13.07	13.52	3¼″
7.81	8.27	8.73	9.19	9.65	10.10	10.56	11.02	11.48	11.94	12.40	12.86	13.32	13.78	3⅜″
7.95	8.42	8.89	9.35	9.82	10.29	10 76	11.22	11.69	12.16	12.63	13.10	13.56	14.03	3½″
8.09	8.57	9.04	9.52	10.00	10.47	10.95	11.42	11.90	12.38	12.85	13.33	13.80	14.28	3⅝″
8.23	8.71	9.20	9.68	10.17	10.65	11.13	11.62	12.10	12.59	13.07	13.55	14.04	14.52	3¾″
8.37	8.86	9.35	9.84	10.33	10.83	11.32	11.81	12.30	12.79	13.29	13.78	14.27	14.76	3⅞″
8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00	4″
7″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.

RIVETS.

Diameters.	Bending Moments in inch-tons.	$\frac{1}{4}''$	$\frac{9}{32}''$	[illegible]	$1''$	$1\frac{1}{32}''$	$1\frac{1}{16}''$	$1\frac{3}{32}''$	$1\frac{1}{8}''$	$1\frac{5}{32}''$	$1\frac{3}{16}''$	$1\frac{7}{32}''$	$1\frac{1}{4}''$	Diameters.
		0.250″	0.281″	0.[illegible]″	1.000″	1.031″	1.063″	1094.″	1.125″	1.156″	1.188″	1.219″	1.250″	
$\frac{1}{2}''$	0.092	0.750	0.844	c										$\frac{1}{2}''$
$\frac{9}{16}''$	0.131	0.844	0.950	1										$\frac{9}{16}''$
$\frac{5}{8}''$	0.180	0.938	1.055	1										$\frac{5}{8}''$
$\frac{11}{16}''$	0.239	1.032	1.161	1										$\frac{11}{16}''$
$\frac{3}{4}''$	0.311	1.125	1.266	1										$\frac{3}{4}''$
$\frac{13}{16}''$	0.395	1.219	1.372	1										$\frac{13}{16}''$
$\frac{7}{8}''$	0.494	1.312	1.477	36	5.250									$\frac{7}{8}''$
$\frac{15}{16}''$	0.607	1.406	1.582	60	5.625									$\frac{15}{16}''$
$1''$	0.736	1.500	1.687	13	6.000	6.188	6.375	6.563	6.750					$1''$
$1\frac{1}{16}''$	0.883	1.594	1.793	76	6.375	6.575	6.774	6.973	7.172					$1\frac{1}{16}''$
$1\frac{1}{8}''$	1.049	1.687	1.898	39	6.750	6.961	7.172	7.383	7.594	7.805	8.016	8.227	8.438	$1\frac{1}{8}''$

						9½"
1.830	1.867					10"
1.850	1.886	1.925	1.959			10½"
1.868	1.905	1.940	1.977	2.016	2.051	11"
1.888	1.926	1.959	1.996	2.034	2.070	11½"
1.913	1.947	1.983	2.020	2.057	2.094	12"
1.936	1.969	2.005	2.040	2.076	2.114	12½"
1.956	1.990	2.026	2.060	2.099	2.134	13"
1.978	2.013	2.048	2.083	2.120	2.153	13½"
2.003	2.037	2.071	2.106	2.140	2.178	14"
2.028	2.061	2.095	2.129	2.164	2.200	14½"
2.053	2.089	2.122	2.155	2.189	2.225	15"
2.081	2.115	2.146	2.179	2.214	2.248	15½"
2.109	2.140	2.172	2.206	2.240	2.273	16"
2.133	2.165	2.199	2.229	2.264	2.297	16½"
2.161	2.192	2.224	2.257	2.288	2.324	17"
2.189	2.221	2.253	2.284	2.319	2.349	17½"
2.219	2.249	2.280	2.313	2.343	2.374	18"
2.244	2.276	2.308	2.337	2.368	2.400	18½"
2.273	2.304	2.334	2.364	2.396	2.428	19"
2.303	2.333	2.363	2.393	2.425	2.445	19½"
2.333	2.362	2.392	2.423	2.452	2.482	20"
2.363	2.392	2.422	2.450	2.479	2.510	20½"
2.393	2.423	2.450	2.478	2.509	2.543	21"
2.425	2.452	2.479	2.509	2.542	2.570	21½"
2.455	2.482	2.510	2.543	2.570	2.596	22"
19½"	20"	20½"	21"	21½"	22"	

2⅛	1.78	2.21		0.197	13/16	0.32
2¼	1.88	2.34		0.215	⅞	0.40
2⅜		2.47	2.97	0.223	15/16	0.47
2½		2.60	3.13	0.231	1	0.55
2⅝		2.73	3.29	0.250		
2¾		2.86	3.44	0.258		
2⅞		3.00	3.60	0.266		
3		3.13	3.75	0.274		
3⅛		3.26	3.91	0.282		
3¼		3.39	4.06	0.291		
3⅜			4.22	0.299		
3½			4.38	0.307		
3⅝			4.54	0.315		
3¾			4.69	0.323		
3⅞			4.85	0.332		
4			5.00	0.340		
4⅛			5.16	0.348		

D
of
co

TABLE XXI.

Sizes of Single Rivetted Lacing Bars for channels of various depths and spaced at various distances.

D = depth of channel, and d = distance between inner faces of channels. **If the value of d lie between the values given, the size of lacing bars is to be taken from the column containing the *next largest* value of d.**

D	Sizes of Lacing Bars.				
	d = D.	d = 1.25 D.	d = 1.5 D.	d = 1.75 D.	$d \geq 2$ D.
4"	¼" × 1⅝"	¼" × 1¾"	¼" × 1⅞"	¼" × 2"	¼" × 2⅛"
5"	¼" × 1⅞"	¼" × 2"	¼" × 2⅛"	5/16" × 2"	5/16" × 2⅛"
6"	¼" × 2"	¼" × 2⅛"	5/16" × 2"	5/16" × 2⅛"	5/16" × 2¼"
7"	¼" × 2⅛"	5/16" × 2"	5/16" × 2⅛"	5/16" × 2¼"	5/16" × 2⅜"
8"	5/16" × 2⅛"	5/16" × 2¼"	5/16" × 2⅜"	5/16" × 2⅝"	5/16" × 2¾"
9"	5/16" × 2⅜"	5/16" × 2½"	5/16" × 2¾"	5/16" × 3"	⅜" × 2¾"
10"	5/16" × 2¾"	5/16" × 3"	⅜" × 2¾"	⅜" × 3"	
12"	⅜" × 3"	⅜" × 3¼"	⅜" × 3½"		
15"	⅜" × 3¾"	⅜" × 4"			

TABLE XXII.

Sizes of Stay Plates to be used in connection with latticing or double rivetted lacing.

D = depth of channel, d = distance between inner faces of channels, t = thickness of stay plate, l = length of same and n = number of rivets on each side of stay plate. If the value of d lie between the values given, the size of stay plates is to be taken from the column containing the *next largest* value of d.

D	Sizes of Stay Plates.														
	d=D			d=1.25D			d=1.5D			d=1.75D.			d=2D.		
	t	l	n	t	l	n	t	l	n	t	l	n	t	l	n
4″	¼″	4½″	3	¼″	4¾″	3	¼″	5″	3	¼″	5¼″	3	¼″	5½″	3
5″	¼″	5″	3	¼″	5¼″	3	¼″	5½″	3	¼″	5¾″	4	¼″	5″	4
6″	¼″	5¾″	3	¼″	6″	3	¼″	6¼″	3	¼″	6½″	3	¼″	6¾″	3
7″	¼″	6¼″	3	¼″	6½″	3	¼″	6¾″	3	¼″	7″	3	¼″	7¼″	4
8″	5/16″	6½″	3	5/16″	6¾″	3	5/16″	7″	3	5/16″	7¼″	4	5/16″	7½″	4
9″	5/16″	7½″	3	5/16″	7¾″	3	5/16″	8″	3	5/16″	8¼″	3	5/16″	8½″	4
10″	5/16″	8″	3	5/16″	8¼″	3	5/16″	8½″	4	5/16″	8¾″	4	5/16″	9″	4
12″	⅜″	8½″	4	⅜	8¾″	4	⅜″	9″	4	⅜″	9¼″	4			
15″	⅜″	10″	4	⅜″	11″	4	⅜″	12″	5						

TABLE XXIII.

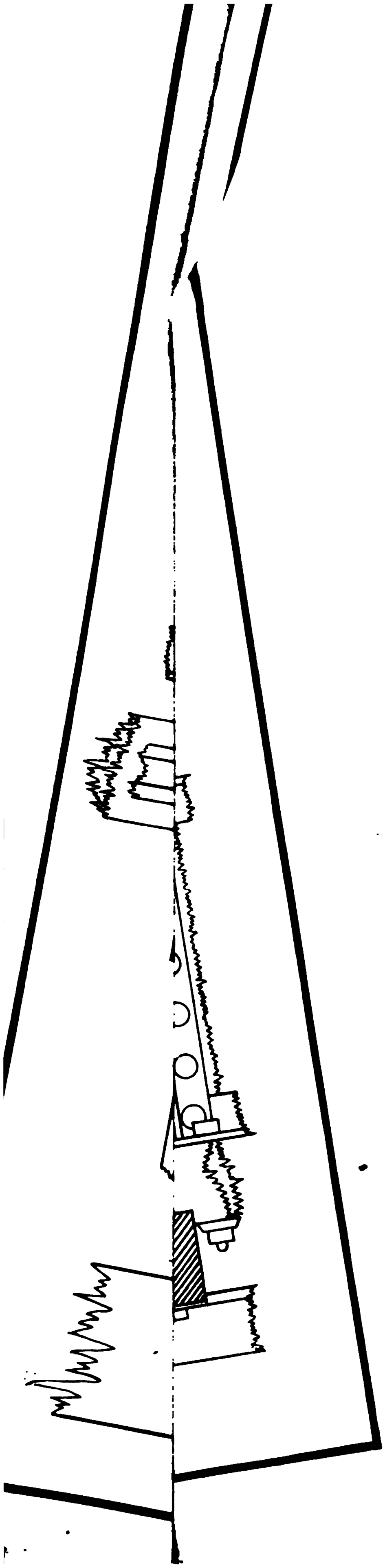
Sizes of Stay Plates to be used in connection with single rivetted lacing.

D = depth of channel, d = distance between inner faces of channels, t = thickness of stay plate, l = length of same and n = number of rivets on each side of stay plate. If the value of d lie between the values given, the size of stay plates is to be taken from the column containing the *next largest* value of d.

D	Sizes of Stay Plates.														
	d=D			d=1.25D			d=1.5D			d=1.75D.			d=2D.		
	t	l	n	t	l	n	t	l	n	t	l	n	t	l	n
4″	¼″	6″	4	¼″	6½″	4	¼″	7″	5	¼″	7½″	5	¼″	8″	5
5″	¼″	8″	5	¼″	8½″	6	¼″	9″	6	¼″	9½″	7	¼″	10″	7
6″	¼″	9″	5	¼″	9½″	5	¼″	10″	5	¼″	10½″	6	¼″	11″	6
7″	¼″	10½″	6	¼″	11″	6	¼″	11½″	6	¼″	12″	7	¼″	12½″	7
8″	5/16″	11″	6	5/16″	11½″	6	5/16″	12″	7	5/16″	12½″	7	5/16″	13″	7
9″	5/16″	12″	6	5/16″	13″	6	5/16″	13½″	6	5/16″	14″	6	5/16″	14½″	7
10″	5/16″	13½″	6	5/16″	14″	6	5/16″	14½″	7	5/16″	15″	7	5/16″	15½″	7
12″	⅜″	16″	8	⅜″	17″	8	⅜″	18″	9	⅜″	19″	9			
15″		20″	8	⅜″	22″	9	⅜″	24″	10						

36½′				35.80	40.00	44.56	49.41	54.59	60.12	65.98	72.21	78.74	85.6[illegible]
37′				35.07	39.21	43.67	48.44	53.54	58.97	64.74	70.85	77.3[illegible]	84.1[illegible]
37½′				34.35	38.38	42.80	47.49	52.51	57.86	63.54	69.56	75.93	82.6[illegible]
38′				33.63	37.66	41.96	46.57	51.51	56.76	62.35	68.28	74.56	81.1[illegible]
38½′				32.97	36.90	41.14	45.68	50.53	55.70	61.21	66.98	73.23	79.76
39′				32.31	36.18	40.34	44.84	49.58	54.67	60.08	65.84	71.92	78.36
39½′				31.67	35.47	39.56	43.95	48.64	53.66	58.94	64.60	70.65	77.0[illegible]
40′				31.05	34.78	39.03	43.12	47.74	52.67	57.92	63.54	69.41	75.66

Y. Nakajima. Del.



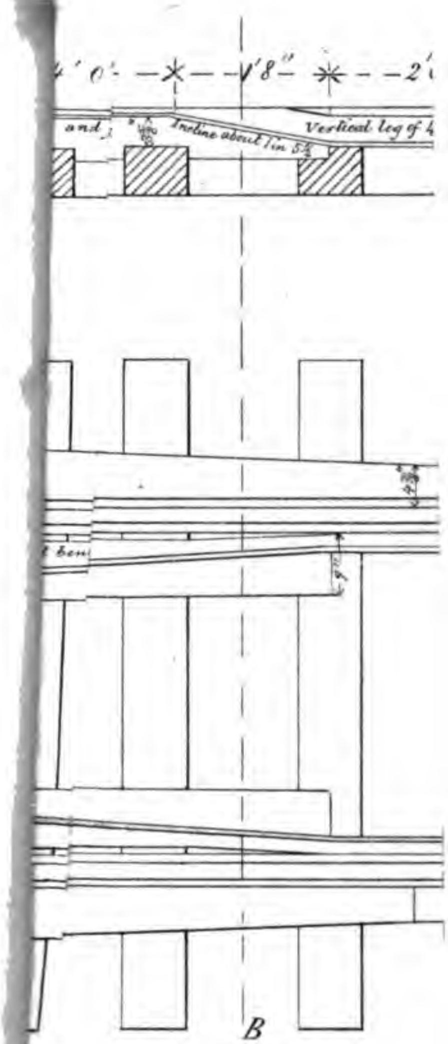
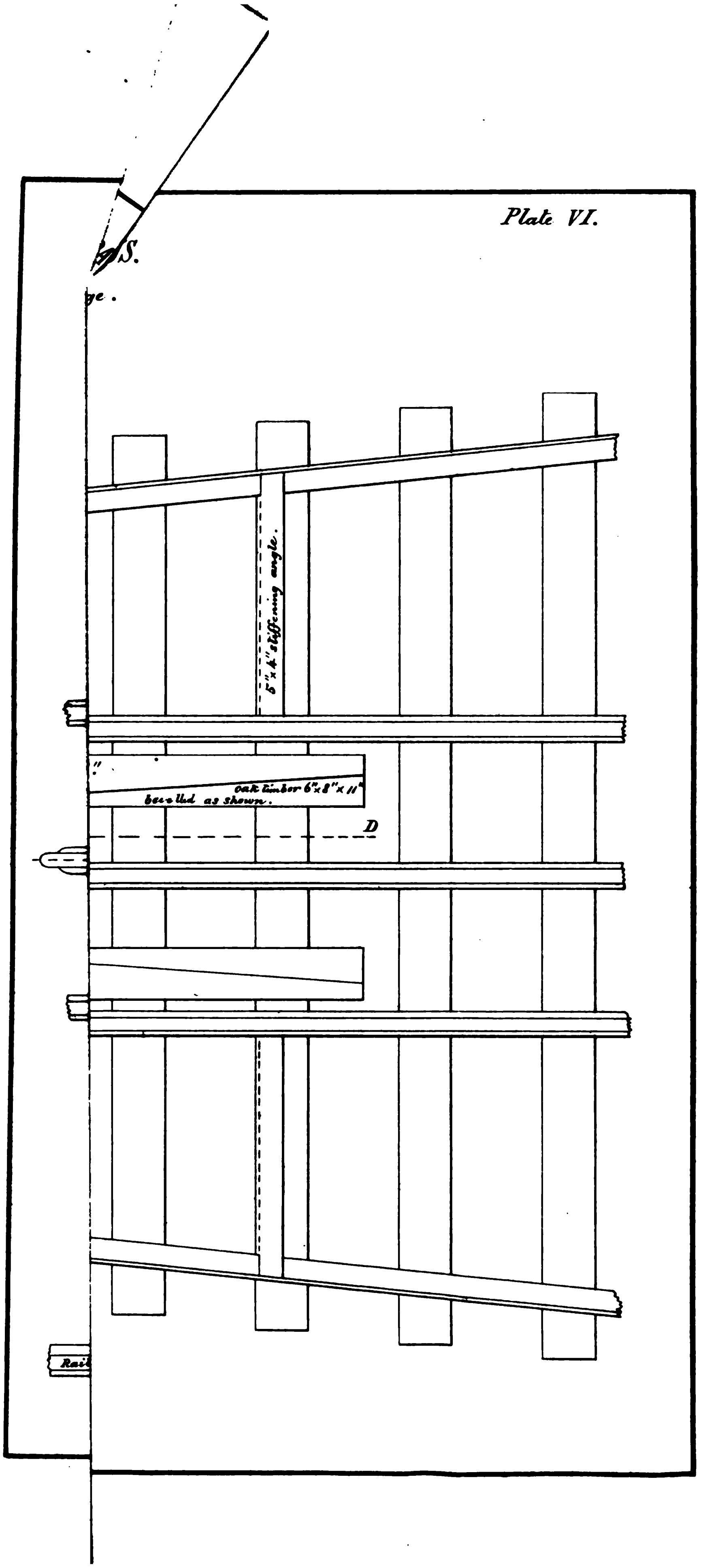
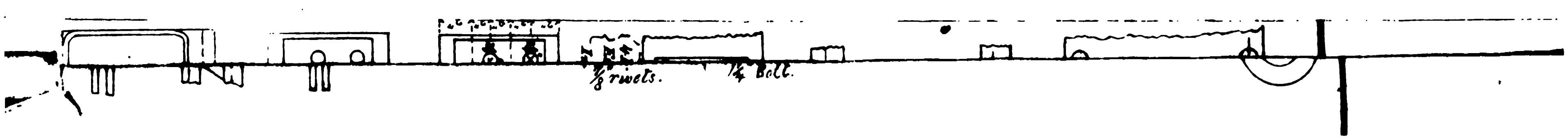

4' 0"
1'8"
and
Incline about 1 in 5½
Vertical leg of 4
B

Plate VI.



S.
ge.
5"×4" stiffening angle.
oak timber 6"×8"×11"
bevelled as shown.
D
Rai



1/8 rivets.
1/4 Bolt.

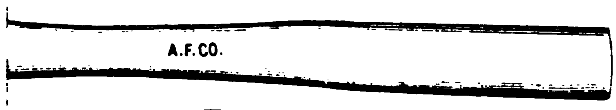
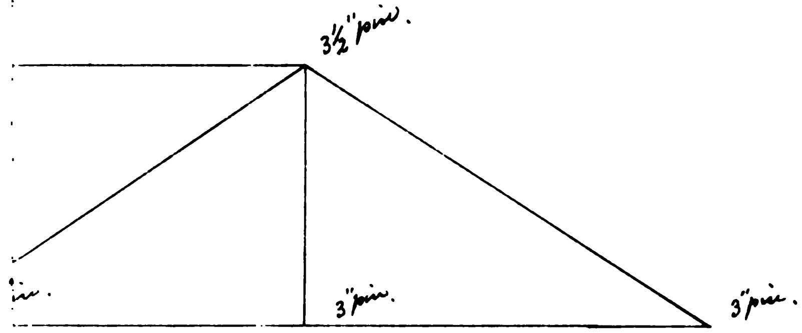


Fig. 19.

Plate XV



Inner bars trussed throughout

Table of Data.

Span	65 ft.
No. of panels	5
Panel length	13 ft.
Depth of truss	8 ft.
Live load	1200# per lin. ft.
Dead load	1020# " " "
Engine excess	82,800#

Plate XVI.

Trus: Diagram

'r a

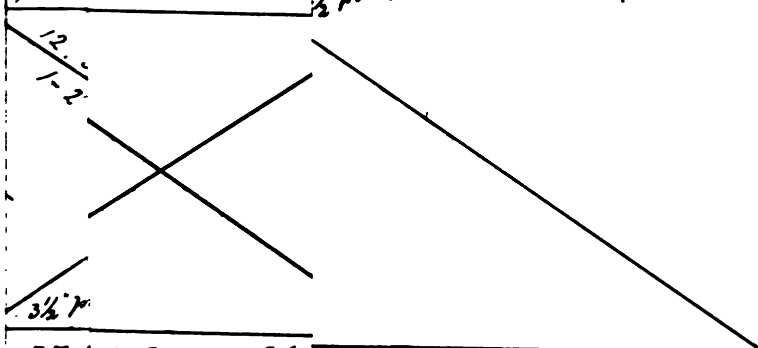
span Pony Truss.

- 10" # [= 11.4 □"

$\frac{5}{16}$" × Pl = 4.69 □"

ot. S_1 = 16.09 □"

7 □" ½" pin.



57. & S. R. = 12.86 hout.

4 - ... = 14.88 □

- 70 ft.

5

.. 14 ft.

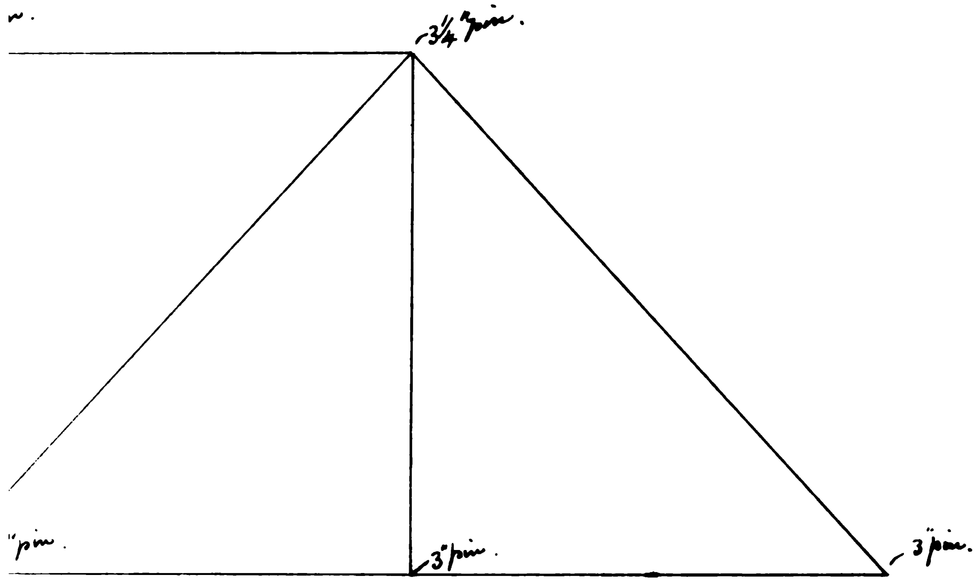
- 9 ft.

1200# per lin. ft.

- 960# .. ..

82,800#

Plate XVII.

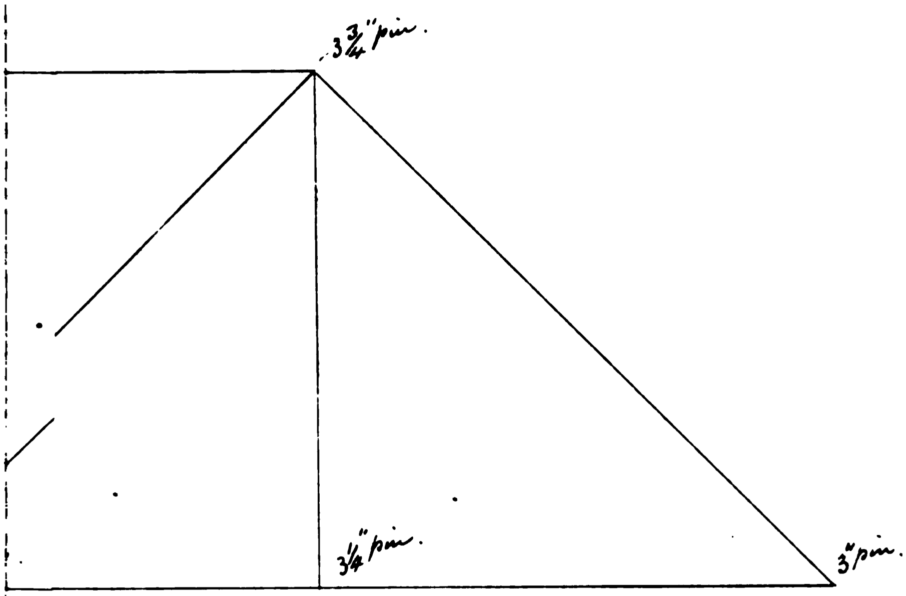


Bottom chord trussed throughout.

Table of Data.

n - - - - - - - - - - - - - - :	70 ft.
of panels - - - - - - - - - -	5.
l length - - - - - - - - - - -	14 ft.
h of truss - - - - - - - - - - -	15 ft.
load - - - - - - - - - -	1200 # per lin. ft.
d load - - - - - - - - - -	1020 # " "
ne excess - - - - - - - - -	82,800. #

Plate XIX

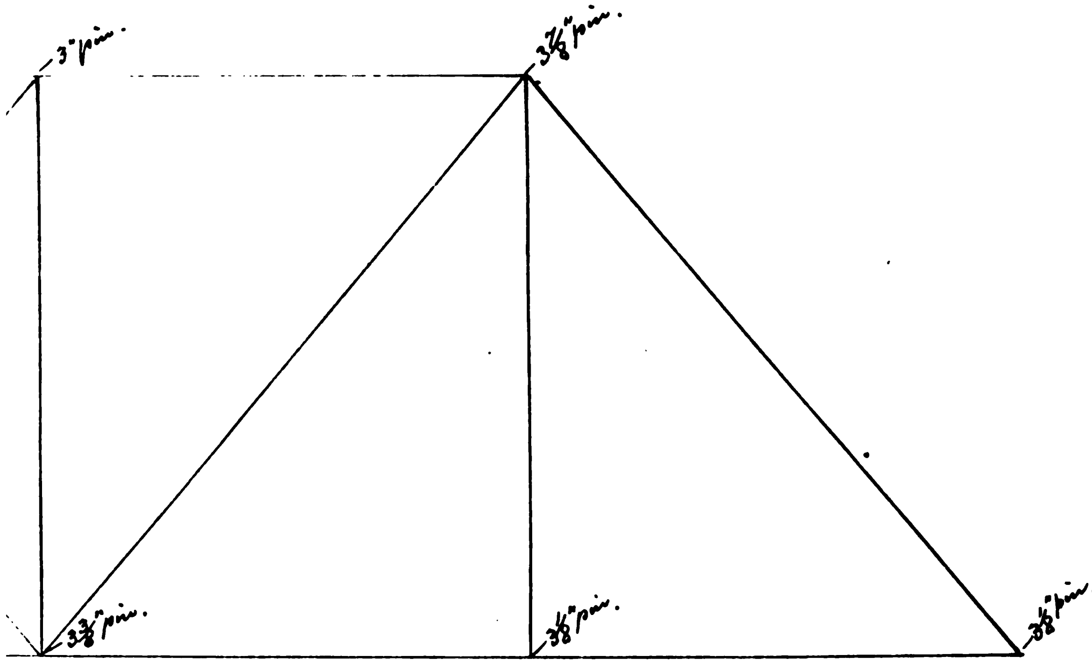


ottcord Trussed throughout.

of .

- - - - - - - - - 90 ft.
- - - - - - - - 5.
- - - - - - - - - - 18 ft.
- - - - - - - - - - 18 ft.
- - - - - - - - - 1200# per lin. ft.
- - - - - - - - - 1080# " " "
. - - - - - - - - 82,800#

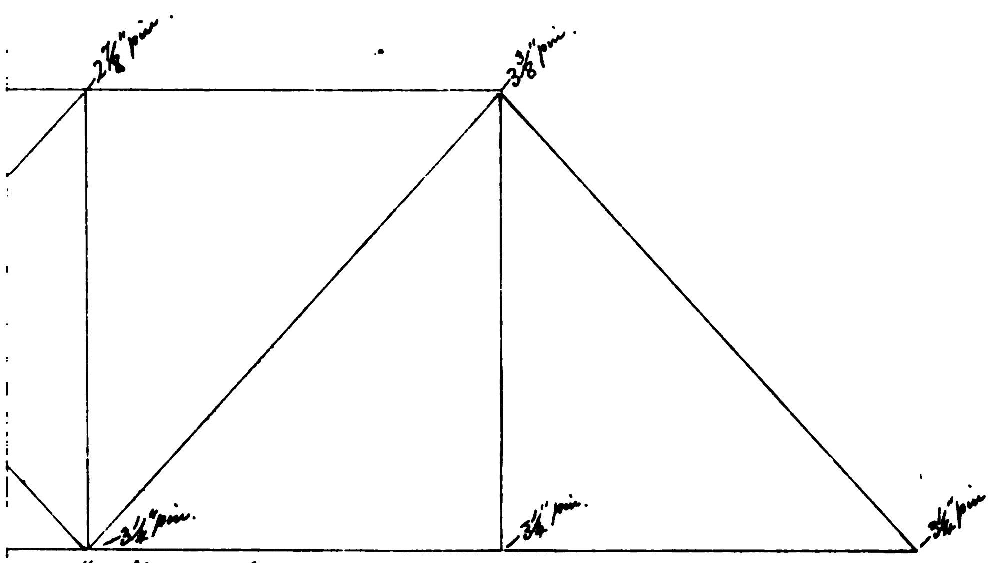
Plate XXI.



Data.

- - - - - - 110 ft.
- - - - - - - 6.
- - - - - - - 18.33 ft.
- - - - - - - 21. ft.
- - - - - - - 1200# per lin. ft.
- - - - - - - - - 1160# " " "
- - - - - - - 82,800#

Plate XXII

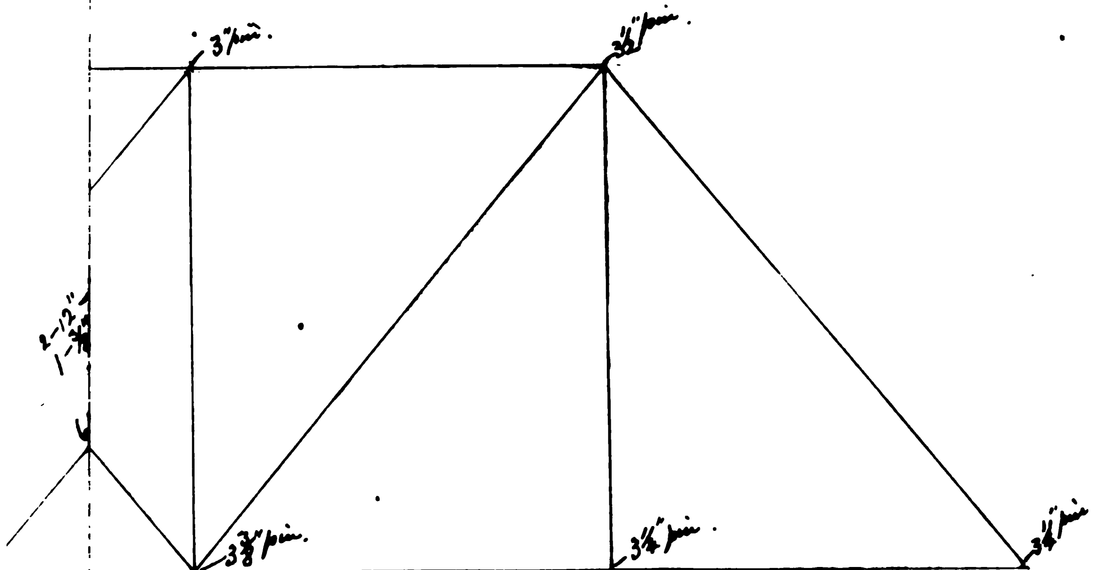


L 8.5# [Throughout
Area of same = 2.8 □"

of Data.

- - - - - - - - - - - 120 ft.
- - - - - - - - - - - 6.
- - - - - - - - - - - 20 ft.
- - - - - - - - - - - 21 ft.
- - - - - - - - - - - 1200# per lin. ft.
- - - - - - - - - - - 1220# " " "
- - - - - - - - - - - 82,800#.

Plate XXIII.

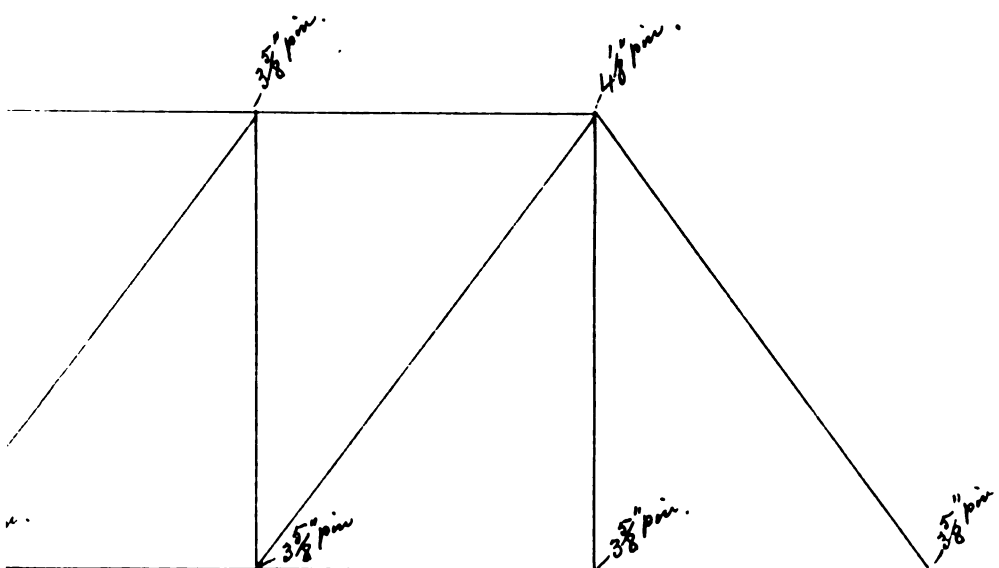


[throughout.
= 2.8 □"

of Data.

- - - - - - - - - 130 ft.
- - - - - - - - - 7.
- - - - - - - - - 18.57 ft.
- - - - - - - - - 22. ft.
- - - - - - - - - 1200# per lin. ft.
- - - - - - - - - 1240# " "
- - - - - - - - - 82,800#

Plate XXX.

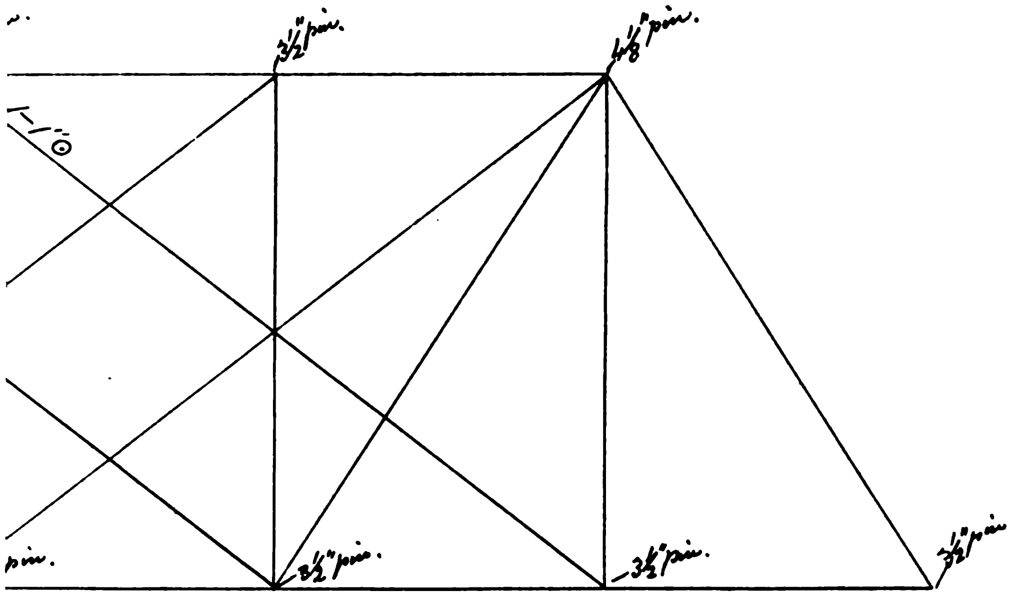


#[Throughout.
= 4.880"

ta.

- - - - - - - 190 ft.
- - - - - - - - 9
- - - - - - - - 21.11 ft.
- - - - - - 28 ft.
- - - - - - 1150# per lin. ft.
- - - - - - 1620# " " "
- - - - - - 85,600#
...om chord - - 430# per lin. ft.
- - - - - - 350# " " "

Plate XXXI.

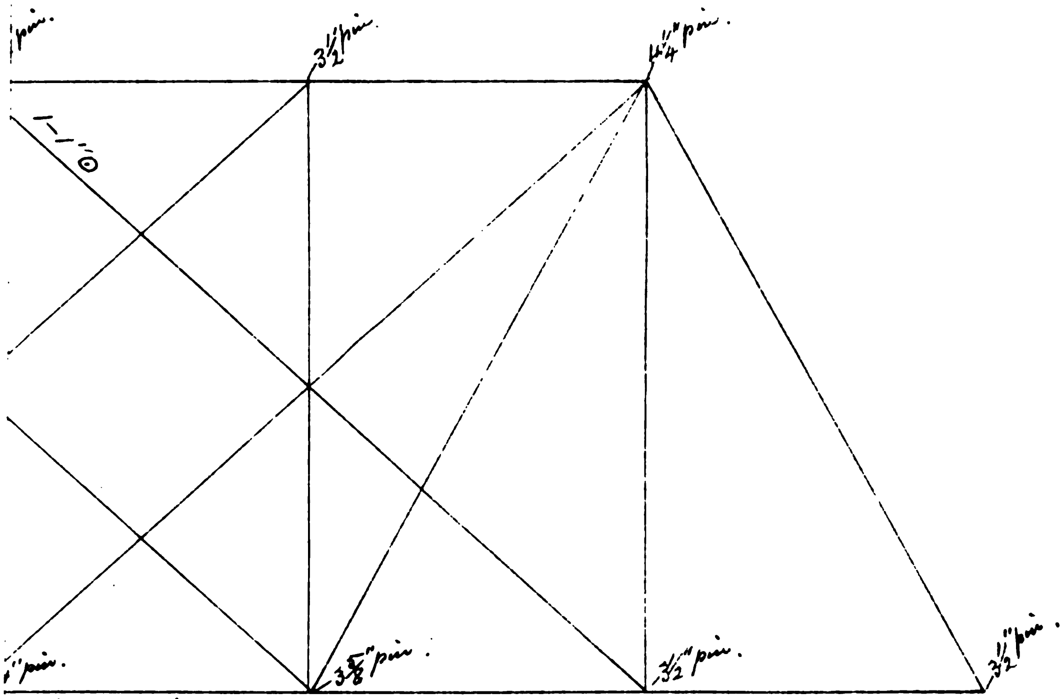


throughout.
= 4.88 □"

Table of Data.

pan -------------------	190 ft.
of panels. -------------	9.
el length. ------------	21.11 ft.
th of truss. -----------	32 ft.
e load ----------------	1150# per lin. ft.
d load ---------------	1600# " " "
ine excess. -----------	85,600#
d load on bottom chord ---	430# per lin. ft.
nsferred load ---------	380# " " "

Plate XXXII.



[throughout.

= 5.4 □"

Data.

- - - - - - - - 200 ft.

- - - - - - - - - - 9

- - - - - - - - 22.22 ft.

- - - - - - - - - 33 ft.

- - - - - - - - 1100# per lin. ft.

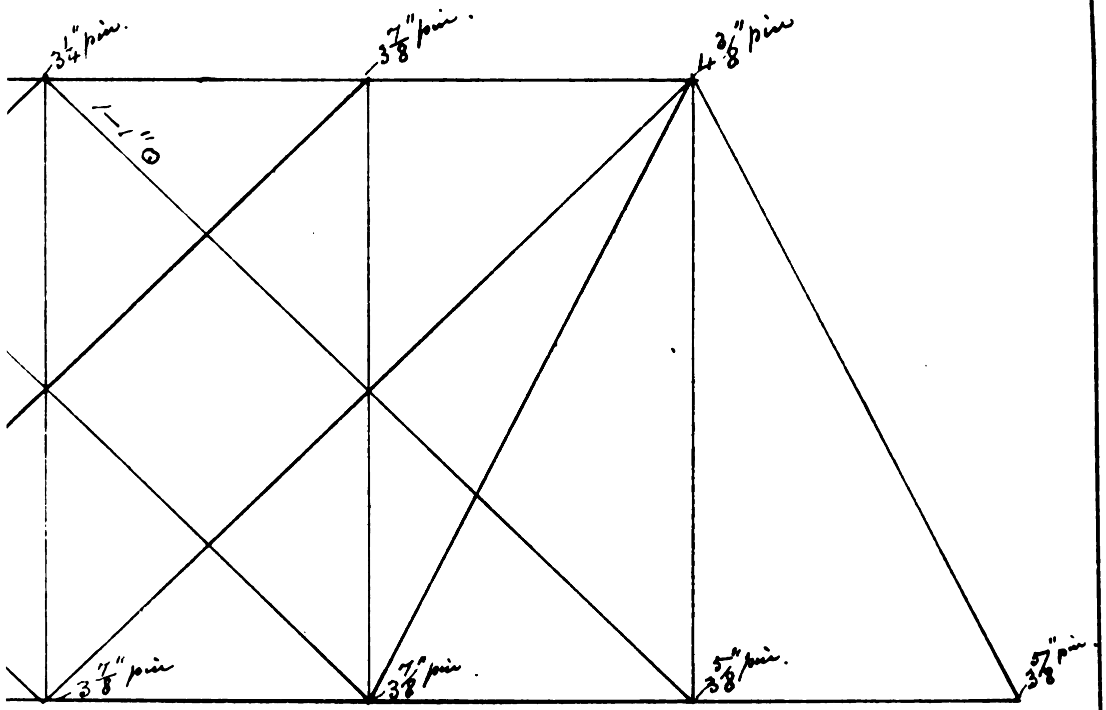
- - - - - - - 1660# " " "

- - - - - - - - 88,400#

tom chord - - - 435# per lin. ft.

- - - - - - - - 380# " " "

Plate XXXIII.



iout

"

-- 210 ft.

--- 10.

--- 21 ft.

--- 34 ft.

-- 1100# per lin. ft.

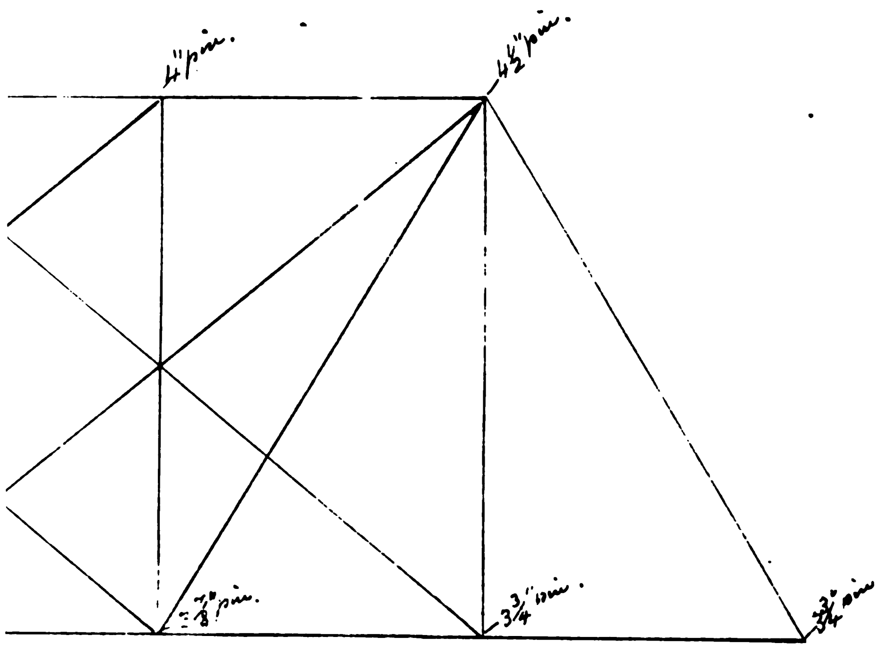
-- 1700# " " "

--- 88,400#

2 -- 440# per lin. ft.

--- 400# " " "

Plate XXXIV.



per lin. ft.
" " "

per lin. ft.
" " "

TG
105
.W12

Waddell, J.A.

System of iron railroad bridges for Japan

134962